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Report
of the
Commissioner Inquiry
Station Square Development
Burnaby, British Columbia

Dan J. Closkey, P.Admin.
Inquiry Commissioner

AUGUST 1988

Province of British Columbia

BUILDER

"If a builder erect a house for a man and do not make its construction firm, and the house which he built collapse and cause the death of the owner of the house, that builder shall be put to death."

— The Code of Hammurabi, *c.* 2250 B.C.

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Province of British Columbia

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COMMISSIONER

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August 26, 1988

His Honour The Lieutenant-Governor in Council

May it please Your Honour:

Under your Honour's Order-In-Council 830, approved and ordered May 6, 1988, I was appointed as Commissioner under Part 2 of the Inquiry Act to report on any and all circumstances surrounding, leading to or having any causal connection with the collapse of a portion of the Station Square Development in Burnaby on April 23, 1988, and to ascertain and recommend measures that could serve to prevent similar tragedies in the future.

I am pleased to advise that I have complied with the instructions, and herewith present my report.

Yours sincerely,

Dan J. Closkey

Dan J. Closkey, P.Admin.
Inquiry Commissioner

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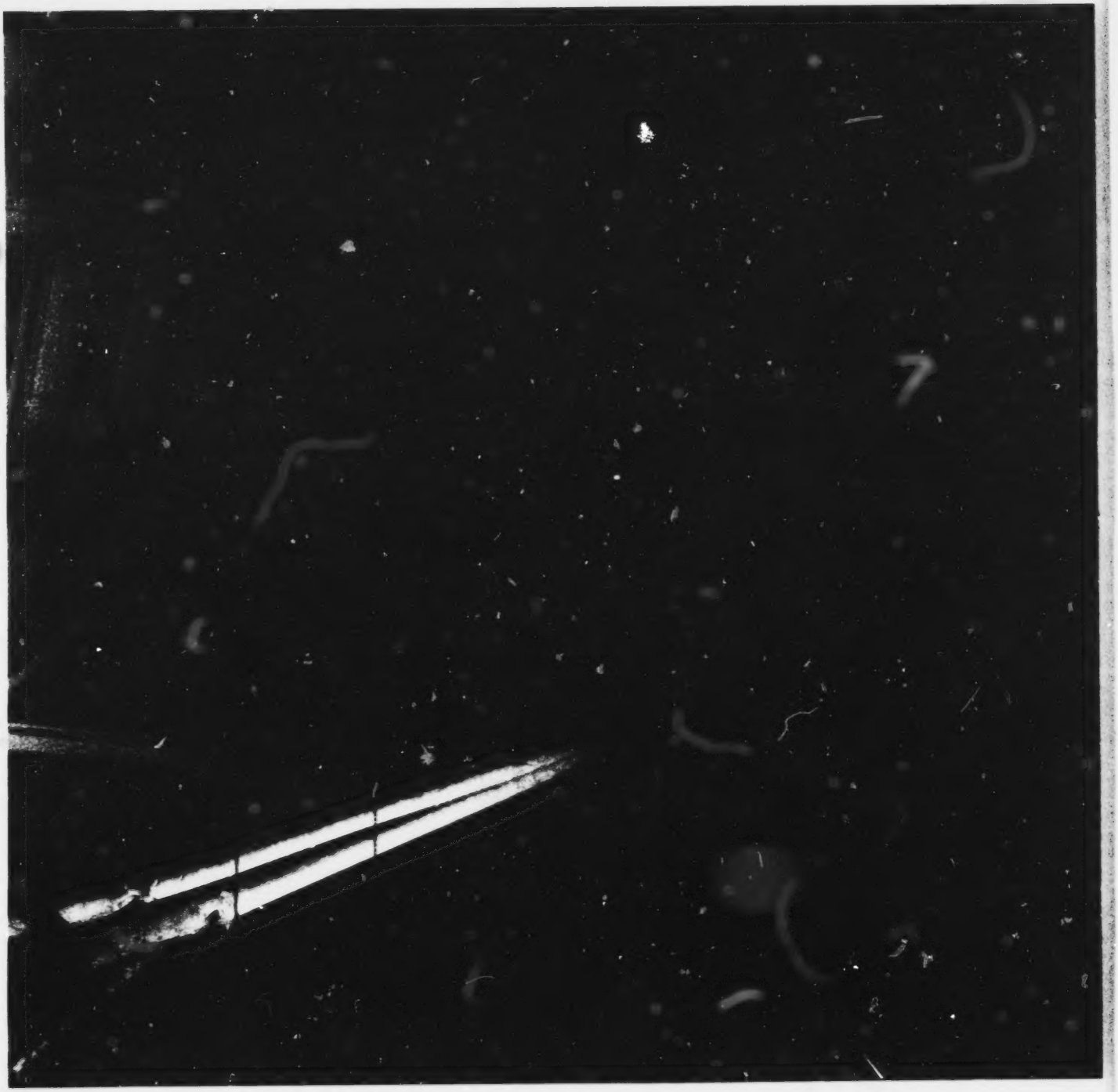

(Trevor Burkitt Photo)

PLATE 1

Summary and Recommendations

On April 23, 1988, Save-On-Foods opened a new store at Station Square in the Metrotown development of Burnaby.

The building, a single-storey steel frame structure approximately two acres (90,000 square feet) in area, is located at the southeast corner of MacKay Road and Kingsway.

It is about 25 feet high and contains rooftop parking for some 235 cars.

At about 9:15 a.m., approximately 6,400 square feet of the roof structure fell into the produce area. Twenty cars parked on the roof fell into the opening and 21 people were injured. There were no fatalities.

The W24 × 76 beam supporting this portion of the roof failed over the column at grid lines S513.33 and W110.84 on the architect's drawing.

The failure involved at least two modes.

Mode A — A beam bending failure with buckling of the lower flange due to its unsupported length.

Mode B — Buckling of the beam-column assembly which, together with Mode A, may have resulted in simultaneous failure in both modes.

Evidence presented to the Inquiry showed that the beam was greatly underdesigned and the beam column assembly lacked essential lateral supports.

Failure was inevitable — if not under April 23 loads, then certainly under full service loads.

When the flange buckled, the column head moved in a southerly direction, about 21 inches. This caused the rupture of a water line that passed through the column, thus warning staff and shoppers below of the imminent collapse.

Collapse was in two stages.

Initially the beam and the beam-column assembly failed and yielded or deformed. Photographs taken between the stages, see page ii, clearly show the severely deformed beam.

The top flange, in general, is still attached to the roof deck, the web has become almost horizontal, the bottom flange has moved, almost the full 24-inch depth of the beam, to the south.

Approximately four and one half minutes later, the roof and cars fell into the store.

No one knows why the roof did not collapse immediately when the beam suffered enormous deformations, but it is likely that the chords of the trusses holding up the roof acted as tendons and the parking deck acted like a membrane and continued to withstand the stresses for this short period of time before it actually fell.

The underdesign was due to a miscalculation in the office of the structural engineer. It was compounded by a similar miscalculation by the structural engineer who was called in to review the structure after deflections were noticed in some beams supporting the travelator in the store.

Facts presented in evidence showed that there is a definite lapse of communication and instruction between the various parties involved in the building process.

The Municipality of Burnaby issued the building permit for the project. The evidence presented to the Commission showed that the municipality's role was strictly administrative, and there was no suggestion that it was deficient in discharging its responsibilities.

The response of emergency teams — fire, police and ambulance — was prompt and efficient.

In addition to the cause of the collapse, the commission studied means of preventing recurrence.

The Association of Professional Engineers of the Province of British Columbia (APEBC) must review some of its procedures with a view to strengthening them.

Contrary to public perception, building inspectors do only a cursory inspection of large buildings. However, municipal departments are the last checkpoint before a building is built and any necessary additional checks must be introduced into the process at either the design stage — architects and engineers — or at the application and permit stage administered by city and municipal building departments.

The responsibility and obligation of the owner received considerable attention throughout the hearings, and in many submissions to the Commission. It is evident that the owners must establish clear lines of communication and responsibility for all the individuals involved in any project.

The matter of public safety is paramount and must be so considered by all parties involved in all building projects for public use. With this in mind, I have made 19 recommendations.

The main recommendations are:

— Provision must be made for the audit of structural engineering drawings and calculations for major buildings selected by municipalities and reviewed by independent engineers, with the cost to be borne by a special levy on municipal building permits. (Recommendation 1.)

— Structural engineers to be required to pass a special examination before becoming professionally qualified, and should be required to carry specified limits of professional liability insurance. (Recommendations 7 and 8.)

— A provincial manual of recommended construction practices and procedures to be developed to clarify responsibilities of owners, architects, engineers, contractors and suppliers in construction projects. (Recommendations 11 and 12.)

— The Minister of Municipal Affairs, Recreation and Culture to appoint a three-member task force to oversee implementation of the recommendations. (Recommendations 18 and 19.)

Other recommendations include:

— Standard documents under the building code to be used throughout the province for major buildings, to assign responsibilities among owners, architects and engineers. (Recommendation 2)

— Architects and engineers to be primarily responsible for ensuring compliance with the building code for major buildings, with building inspectors to play a secondary, administrative role. (Recommendation 3.)

— To strengthen standards within the engineering profession, engineering firms to be registered with the APEBC, in addition to the registration of individual engineers. The association to be authorized to prescribe a minimum fee schedule, to be approved by Cabinet. (Recommendations 5, 6 and 9.)

— Provincial standards of practice to be established for major building design, drawings and calculations. (Recommendation 10.)

— Municipal by-laws to be updated to refer to the current B.C. Building Code. (Recommendation 4.)

— Steel industry construction manuals to be revised to provide more accurate assistance to engineers. (Recommendations 13, 14, 15, 16 and 17.)

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Preface

This Inquiry was appointed by Order-In-Council on May 6, 1988, to inquire into the collapse on April 23, 1988, of a portion of the roof over the Save-On-Foods store in Burnaby. The store was part of the Station Square Development at Metrotown.

The terms of reference of the Inquiry are reproduced in full in Appendix I to this report. In summary, they are divided into two components. First, I am to determine the cause of the collapse. Second, I am to make recommendations to prevent a recurrence in future.

Commission Procedures

The Commission held 10 days of hearings between May 30 and July 6. A total of 47 witnesses gave sworn evidence at the hearings. The president of the B.C. and Yukon Building Trades Council presented a submission and, as a result of this presentation, five workers appeared at an evening sitting.

Fifty-five exhibits were filed, including 34 written submissions. Copies of most of the written material were available for public inspection at the Central Park branch of the Burnaby Public Library.

Appendices VI through IX set out details of the hearings, witnesses, written submissions and exhibits.

Appendix XIII is a conversion table as all calculations in the report are in Imperial units.

This report contains a number of technical expressions, defined in the glossary, Appendix XV.

The Commission did not find it necessary to issue any subpoenas to compel attendance of witnesses, and the voluntary cooperation of all involved was extremely helpful to the Inquiry.

Proposed rules of procedure were circulated at the first hearing, setting out the time frame for written submissions, rights of examination and cross-examination, participation of legal counsel and so on.

I am pleased to report that, considering the complexity of the legal issues raised by the roof collapse and the resulting potential for drawn-out and costly public hearings, the hearings proceeded with efficiency and dispatch. I am grateful for the assistance of all participants, and their advisors, in this regard.

Evidence produced at the hearings was supplemented by interviews by me and my staff with witnesses, and others who came forward with information. Professional engineering organizations in Canada and the U.S. provided valuable and timely assistance.

As well, Commission personnel on numerous occasions visited the Station Square Development site and the warehouse where the rubble from the collapse was stored.

Organization of Report

This report is organized along the lines of the terms of reference. Part I — "Collapse" — first describes eyewitness accounts of the collapse and, in general terms, the project.

Chapters 2 through 5 describe structures generally, the building in particular, and analyze the causes of collapse and describe how the error was made that led to the collapse.

Chapter 6 describes the role of contractors in the project. The independent second review that was carried out on the structure is described in chapter 7. Part I concludes with a summary in chapter 8.

Part II, "Prevention," is a wider-ranging discussion of public policy in the province as it relates to the design and structural integrity of buildings. Following a brief overview in chapter 9, project organization and responsibilities, in general terms, are summarized in chapter 10.

The responsibilities of municipalities are reviewed in chapter 11. Recommendations are proposed to assist this level of government in discharging them.

Project owners play a central role in development. Trends in construction organization are reviewed in chapter 12, with particular emphasis on "fast-tracking" of projects, bidding for professional fees and fragmentation of responsibilities.

Chapter 13 briefly reviews the role of architects in the industry.

A focal point of this report is chapter 14 — Responsibilities of Engineers. The roof collapse at Station Square has raised questions about the regulation of the engineering profession in the province. Recommendations there are intended to maintain adequate engineering standards.

Chapter 15 — Construction Practices, Procedures — is aimed at pulling the industry together with common rules governing project organization. Chapter 16 outlines suggestions for the structural steel and steel decking industries.

Guidance for the implementation of the recommendations in this report is provided in chapter 17. The concluding summary of Part II is set out in chapter 18.

Public Safety

Throughout this report, the prime objective of the recommendations is to safeguard the public. However, all concerned in a major construction project have conflicting objectives, to varying degrees.

Owners are primarily motivated by a satisfactory return on investment. Contractors and suppliers likewise are profit-oriented.

The professional engineers and architects have dual responsibilities. On one hand, by training and through professional ethics, they have a duty to maintain a high calibre of service to the public. In the context of building design and erection, this translates into the protection of public safety.

On the other hand, professional consultants operate businesses in a commercial world. They, too, require a reasonable stream of revenue to survive.

In the middle stand the regulators: municipalities, responsible for enforcing building standards, and the professional associations, for maintaining professional standards.

Ultimate responsibility for public safety lies with the government of the province. The recommendations in this report are aimed at assisting the government in discharging its mandate.

Acknowledgements

The Commission wishes to acknowledge the prompt response received from the many organizations and individuals in Canada and the U.S. to requests for information which provided the Commissioner with a much broader perspective as to how other organizations operate in other parts of North America.

The many excellent briefs submitted to the Commission from interested individuals and associations supplied us with a great deal of information that simplified our task in this complex matter.

These submissions cover a wide range of subjects pertaining to the building industry in British Columbia. The Commission is appreciative of the many hours of research and thought that was put into their preparation.

I thank the lawyers and engineers involved in the proceedings. With their help, the Commission was able to complete its public hearings in only 10 days.

The cooperative manner in which witnesses came forward voluntarily to provide evidence and information to the Commission was, indeed, helpful. I also commend the witnesses for the honest and forthright manner in which they presented their evidence.

The public hearings were broadcast live, and the efforts by Greg Bosworth and West Coast Cablevision Ltd. in televising our proceedings to homes in much of the Greater Vancouver area are worthy of mention. I also thank Jim Howe for his advice and superb sound system.

The many support functions provided by various provincial government agencies such as supply of stationery, office furniture and accommodation, computers, telephones, etc. has been of the highest order. Without this excellent support, the Commission would have found its task much more difficult.

The advice and assistance provided by Susan Hix and the staff at the Queen's Printer in printing this report are greatly appreciated.

The Commission and its advisors received continued support throughout the hearings from Ms. Pamela Chow, receptionist; and from Mrs. Denise Kellett, Executive Secretary, who was of great assistance to the Commission in recording the exhibits, maintaining files on all submissions and correspondence, and in the production of this report.

Commission Counsel, Mr. Richard Campbell of the law firm Campney & Murphy, was of great assistance to the Commission throughout the Inquiry. Mr. Campbell also provided valuable guidance in the public portion of the Inquiry, the examination of witnesses, assessing much of the information, and in the preparation of this report.

The expertise of the Commission's two engineering advisors, Mr. C. Peter Jones of Jones, Kwong, Kishi, who is also vice-president of the Association of Professional Engineers of the Province of B.C., and Dr. Noel Nathan, Professor of Civil Engineering at the University of British Columbia, has been invaluable to the Commission in its investigation.

The Commission wishes to acknowledge the dedicated effort and support of Mr. Charles La Vertu who acted in the capacity of Commission Secretary and Administrator.

His expertise in the areas of communications and administration is largely responsible for the satisfactory media coverage and the quick conclusion of the Commission. He also carried out the preparation and production of this report, and many other assigned tasks in an exemplary manner.

PART I

COLLAPSE

CHAPTER 1 — *Market to Misfortune*

The Opening

It was opening day for the new mega Save-On-Foods store at Station Square in Burnaby.

The first customers arriving by car were directed by commissionaires to the parkade on the roof of the store.

There was a light drizzle and some customers complained of having to park on the roof when covered parking was available in the adjoining three-level parkade. The third and fourth parking rows were about half full when customers were allowed to park elsewhere.

Following an official opening ceremony, the store was quickly filled with an estimated 600 senior citizens attracted by special discounts. About 370 employees were on hand to greet them.

Minutes later, at about 9:15 a.m., a section of the roof collapsed and 20 cars parked above fell into the produce department. No one was killed but 21 people were injured.

Save-On-Foods is the tenant, and was not involved in the construction of the building. The supermarket is operated by the Overwaitea Division of Jim Pattison Industries Ltd. There are 57 stores in the chain including nine operated under the Save-On-Foods banner.

The Collapse

William Baxter was picking up an English cucumber, Eva Howie and her friend were buying fish and Arthur Pastro was checking the price of lettuce when they were asked to leave the store.

Shoppers heard a loud cracking and banging noise. Many thought it was a display that had tumbled to the floor. At the same time, an overhead pipe burst open and sprayed water over the bulk foods area near the cheese delicatessen counter.

Employees started to usher the confused and reluctant customers out of the area.

On the rooftop parkade, photographer Greg Kinch was putting his camera gear away when he heard a crunch which sounded like a large truck hitting a cement barricade. He turned around to see what had happened and saw two rows of cars "bouncing up and down."

In the store, Burnaby Mayor Bill Copeland, on hand for the opening ceremonies, saw that a ventilating duct had sagged, and water running.

Utility clerks began mopping up the water.

On hearing "popping, cracking noises," the mayor, a former fire fighter, suggested that the employees leave the area because "something may come down on you."

"I could see the distortion of the post, and looking up from that you could see that there was distortion above, on the beam, and I knew at that moment that something disastrous was going to happen."

Meanwhile, everyone was being told over the public address system to move to the front of the store, and Copeland immediately began helping store employees move people away from the area.

When customers were told over the P-A system to move to the front of the store, retired engineer Ernie Mason told his wife that they'd best get out of the store. They went through check-out counter 13 which stamped the time on their bill: It was 9:11 a.m.

On hearing the initial appeal over the public address system, Jim Boyson, vice-president for Overwaitea, headed for the produce area to find out what was happening.

"I noticed the beam was rotating, and there was a leak in the pipe in the ceiling

Plate 2: Shows parking deck sidewalk still intact — cars and large portions of the roof structure as they came to rest on the floor.

Plate 3. Shows beam in post collapse position with flange buckled and web slipping evident. (Photos by Chester Machniewski)

in the location. I knew exactly what was going on. I knew we had a cave-in problem," Boyson said.

Everyone was then told over the P-A to leave the store immediately.

Trevor Burkitt, an amateur photographer and photo-technician at another Save-On-Foods store, was drawn to the area by the running water and started taking pictures when he saw "that there was something terribly wrong."

He was right underneath the beam and zoomed in for a close-up: "It was then that it started to appear to me that there was a major problem, and that I was really in the wrong place."

Burkitt heard a crack "and I turned around and aimed my camera up at the roof, and then there was a tremendous roar as the roof sagged and just fell right in."

Estelle Birch and a girlfriend watched the ceiling "come down very slowly, and (we) got out of there fast. They gave us a choice to go out the left or the right, and my girlfriend and I chose the right which was a good thing because everything was flying at the other end. And that's when that air hit everybody, just like a tornado going through."

Elsie Barber said the gust knocked her down "and then all I knew, people picked me up, then I looked, my feet were wet, but my shoes were off my feet."

The blast of air and the water on the floor caused Molly Jensen to fall on the floor where she broke her elbow.

"The gush of air that come out of the building, with a nutmeg smell, blew us out," said Bud Smallwood.

He was helping pick up an elderly lady "when another woman fell over the top of both of us. I could see that there was going to be a pile-up in front of the door, so I stayed there to help the people, drag them out."

The exodus was orderly and everybody was well behaved. Testimony showed that the quick and courageous action on the part of the employees and the mayor saved many customers from injury caused directly by the collapsing roof.

Utility clerk Larry Nichols, who was in the danger zone cleaning up, did not get out in time, and his hips were pinned under bent steel and broken concrete. Firemen and emergency health services personnel used two pallet jacks to remove him from under the rubble. He suffered a crushed pelvis.

The seniors who lost their cars in the collision were upset at having to accept considerably less than they feel their vehicles were worth.

"We had no way to protect ourselves, and I think they should look after us," said Annabel McDaniel who did not have comprehensive insurance. "I went in with a car; I don't see why I can't come out of there with a car."

The Project

Land assembly, consolidation and rezoning for the $100 million Station Square project at Metrotown was started in 1985 by Georgilas Investments.

Urban Design Group Architects was retained to prepare preliminary conceptual design drawings of a community shopping centre.

Early in 1986, Wesbild Enterprises Ltd. and Station Square Developments Inc. bought the project and retained Urban Design as architects and project coordinator from the planning stage through to construction.

The project is to be a mixed-use shopping centre complex and will include a hotel, apartment towers, commercial retail units, theatres, office buildings as well as the store which is to be the flagship of the Save-On-Foods chain.

Urban Design solicited proposals for the mechanical, electrical and structural engineering, and selection of the consultants was made by the owners.

Eight bids were received on the structural engineering work and Tamm Tacy and Associates, the third lowest bidder, was chosen.

Its bid was in the amount of $20,500 for the $5.4 million Save-On-Foods building. This fee

was subsequently negotiated down to $17,000 by the Wesbild development manager.

Wesbild/Station Square pre-qualified the contractors as to ability to build a project of this size and financial ability, and chose Amako Construction Ltd. which was the lowest of six bidders.

Amako in turn hired the sub-contractors including Empire Iron Works to provide and erect the steel. Empire hired Great West Steel Industries to design and supply the open web steel joists and Dowco Consultants Ltd. to detail the connections.

Original plans for the Save-On-Foods building did not include parking on the roof, and in December, 1986, one of the contractors bidding for the contract suggested the rooftop parkade during prequalification.

Both Urban Design and Tamm Tacy reviewed the proposal and reported that it was feasible. Discussions were held with Save-On-Foods and the idea was adopted.

A travelator was to take people from the floor of the store to the roof.

All plans, drawings and specifications were finally signed off by Save-On-Foods on July 16, 1987.

Tamm Tacy provided the Municipality of Burnaby with a letter indicating that the building construction would be supervised by a structural engineer.

Building permits were issued for a fee of $30,235, and construction started July 17, 1987.

The sidewalk on the parking deck was originally five feet six inches wide as shown on architectural drawing A5.5. On July 6, 1987, revision six was issued extending the sidewalk three feet on each side for a total width of 11 feet six inches.

Drawing S5.5 provides an edge detail of the sidewalk showing the sidewalk six inches above the finished roof-parking surface with an edge thickness of 10 inches and a center core of six inches of Styrofoam insulation.

Amako proceeded with the rooftop construction and poured the three feet extensions on each side of the sidewalk in solid concrete.

The designer, an engineer-in-training at Tamm Tacy, said the edge detail on drawing S5.5 should have prevailed, and the additional weight would have been minimal.

Tamm Tacy would not have agreed with the addition of six feet of concrete six inches deep had they been consulted on that point.

The additional weight of the sidewalk supported by the beam became 24,500 pounds, a significant increase in weight and a contributing factor to the eventual collapse.

The extra weight, although in place, was ignored by another firm of consulting engineers, The MSS Group, that did a subsequent structural review.

Meanwhile, Great West Steel started design of the open web steel joists and in September and October was requested by Tamm Tacy to add bottom chord extensions to 29 locations.

There was a note on one of the structural drawings requesting that bottom chord extensions also be placed where applicable.

GWS informed the consultants that the extensions would involve extra costs, and the consulting engineers decided that these would not be required to provide stability for the columns.

"The omission of the extension at the critical column location had a very significant effect," said John Rapson, the consulting engineer in charge of the remedial work in the store.

"It's my belief that had the extension been in place, the beam would have been still very severely over-stressed, but it would probably not have collapsed on that day. We may very well have been shopping in that store today unaware of the level of distress in the roof structure above us."

There were no problems until November, 1987, when a beam in the area of the travelator deflected about two inches, tilting the housing.

The owners asked Tamm Tacy to review the problem and remedial work was recommended and done. Several beams were

strenghtened, some columns were added; a new reinforced concrete beam constructed; and some details modified.

Station Square decided to get an independent opinion of the structure and hired MSS to review this area and to look at the building and the design drawings for problems.

In a Dec. 24, 1987 letter, MSS submitted a report identifying the beams that they found to have "insufficient capacities;" discussed the remedial action carried out, or proposed, by Tamm Tacy; and agreed with it.

Letters from both firms confirmed that the remedial work had been done.

In March the tenant, Save-On-Foods, became concerned about deflections of certain roof beams and hired The MSS Group to conduct a design review of the entire building.

"We didn't set a time limit on it, nor did we set a dollar value on what we were prepared to pay," said Boyson. "The main thing was to get a good second opinion on the safety of the building."

In a report dated March 30, 1987, MSS reported that two beams had "insufficient capacities." One was the W24x76 that later collapsed. The report recommended remedial measures, and Wesbild, the architects and both engineering firms initiated steps to effect this remedy.

The following day, the owners were advised that the remedial work would not be required because a mill certificate provided by Empire Steel to Tamm Tacy indicated a yield strength of 55,700 per square inch (psi). The design strength was 44,000 psi.

"On the basis of this information, we conclude that the beams in question are at least 26 per cent stronger than the specified strengths and are satisfactory to resist the design loadings imposed," said Tam London of MSS in an April 7 letter to Overwaitea Foods.

"This new information is a blessing for all concerned."

Construction was completed on March 18. Tamm Tacy informed the Municipality on April 14 that the building had been built according its drawings and specifications and to the National Building Code of Canada.

A temporary occupancy certificate was issued by the Municipality on April 22.

On April 23, shortly after the official opening, the beam and an area of approximately 6,400 square feet collapsed.

CHAPTER 2 — *Structures — General Background*

Loads and Capacities

Every structure and every structural member (beam column, footing, connection, etc.) is subject to loads which create stresses or moments. If these exceed the capacity of the member, then failure occurs.

One type of load is tension. If a rod one square inch in area carries a 100-pound load, the tensile stress is 100 pounds per square inch, or 100 psi. If the load is high enough, a tension member will fail by tearing.

FIGURE 1

If instead of pulling we push, then we have compression. A load of 1,000 pounds on a column two inches by two inches creates a compressive stress of 1,000 divided by four, or 250 psi. If the load is high enough, the column will fail by crushing, as occurs when we press hard on a lump of sugar.

FIGURE 2

If a column or compression member is slender, then before the load gets large enough to cause crushing, the member will fail by springing out sideways or "buckling." For any given cross-section of a member, the carrying capacity decreases as the unsupported length increases.

FIGURE 3

Another type of loading occurs when a force acts at right angles (or, indeed, at any angle) to the axis of the member. Bending is then created in the member. If a 1,000-pound load acts at the free end of a six-foot-long beam (Figure 4), the effect, called the moment, increases toward the support where it is maximum.

At the support, the bending moment (often designated M) is

M = 1,000 pounds × 6 feet
= 6,000 foot/pounds

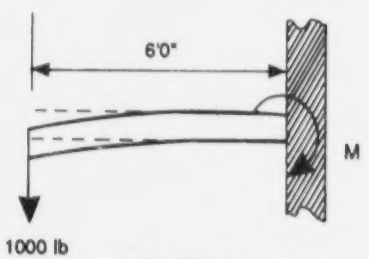


FIGURE 4

Engineers use the term "kip" for 1,000 pounds so we can say M = 6 Ft.-Kips. As shown in the illustration, the beam bends concave downward. The top of the beam is in tension and the bottom is in compression.

A specific structural member when subjected to a specific load develops a specific tension, compression or moment. The member also has a specific tension capacity, compression capacity and moment capacity. If the load effect is greater than the capacity, a failure occurs.

In addition to tension, compression and moment, loads may cause other effects such as shear, web crippling or torsion.

Our calculations indicate that these latter effects were not critical at the time of collapse. They are not discussed in this report. For calculations, see Appendix III *Interim Report on the Cause of the Collapse.*

Engineers speak of live loads and dead loads. Live loads are those that are applied as the structure is used — such as people, furniture and parked automobiles. Dead loads are those that just lie there — the weight of the structure itself and permanently applied material.

Structural Steel Members

The beam that failed was a steel "wide flange section."

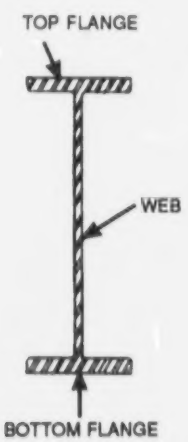


FIGURE 5

It consists of a thin vertical element called the web, and top and bottom horizontal elements called the top flange and bottom flange respectively.

A wide range of sizes is available. The subject beam is referred to as a W24 × 76 — that is a wide flange section 24 inches deep, weighing 76 pounds per linear foot.

At the failure location, the beam ran over a column. Here it was bent concave downward. The top flange carried a tensile force and the lower flange an equal compressive force.

If both flanges had been laterally restrained, failure would have occurred either by tearing of the top flange in tension or by crushing of the bottom flange in compression. The capacities in compression and tension would have been equal or nearly so.

The bottom flange did not have lateral support, allowing it to spring out sideways or buckle. The capacity of the lower flange was, therefore, reduced and the capacity of the beam to resist moment correspondingly reduced.

The column member per se was a steel hollow square section 12 × 12 inches with one-half-inch thick walls.

However, because the loads from the joists were transferred to the top of the beam which sits on the 12 × 12-inch section, the actual column mechanism is complex and includes the depth of the beam. A simplified view is illustrated in Figure 6.
As can be readily visualized, the two-tier column has a relatively low capacity to resist buckling at the bottom of the beam web. (Point X)

Modes of Failure

Failure obviously occurred, and the Inquiry attempted to determine what form it took, and where it started. It is apparent that two forms or modes of failure were involved:

— a buckling of the two-tier beam column system;

— a buckling of the compression-loaded bottom flange leading to a bending failure.

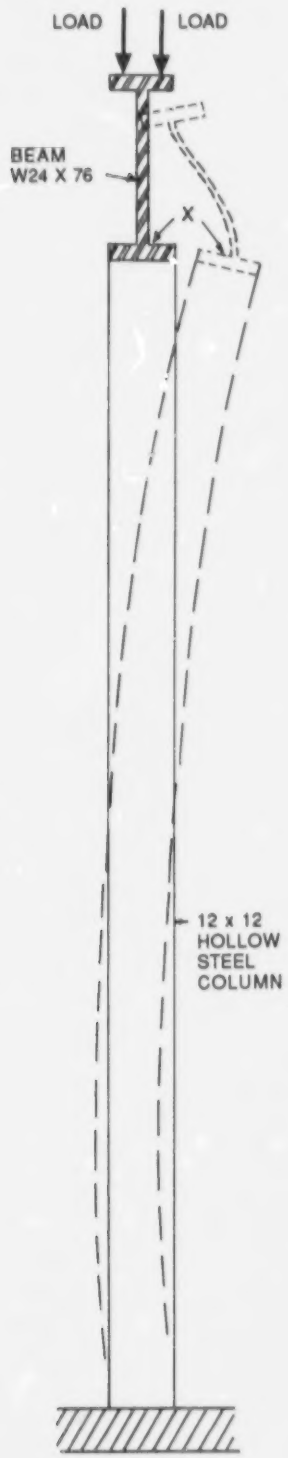


FIGURE 6

Although both modes were present when the photograph (see Plate 1) was taken, either one or the other was the primary mode, or both could have occurred simultaneously.

Before we investigate the primary mode of failure, let us look at the process of *design* and how it differs from the process of *investigation*.

Structural Design Process

The structural engineer calculates two values: (1) the stresses or moments created by the loads; and (2) the stress capacity or moment capacity of the member.

He does this for each load condition which may act on a member. The layman may believe that these are precise values. In fact, they are not. They are probabilities, or ranges of probabilities.

For example, the B.C. Building Code gives a live load for parking decks of 50 pounds per square foot (psf). This is, in the opinion of those who drew up the code, the highest load to be expected during the life of the project — say 50 years.

There may never be more than parked cars on the structure, which may not exceed 20 psf. In 30 or 40 years, someone may drive a very heavy vehicle on to it, or a huge crowd may gather to witness some event and create loads greater than 50 psf.

The code simply gives the most probable maximum load, shown as L_p on Figure 7. The actual load may vary between L_l and L_m, the least and the greatest.

Then the engineer calculates the capacity of the beam, column, or slab. This is C_p. Again, it is not the precise truth. At this stage, the steel has not been bought, nor has the concrete been poured, and the exact capacity or strength is not known. Small errors will be made in construction, the dimensions will not be exact.

The engineer's calculations are not expected to be absolutely precise, and environmental conditions such as temperature variations and foundation movements will cause

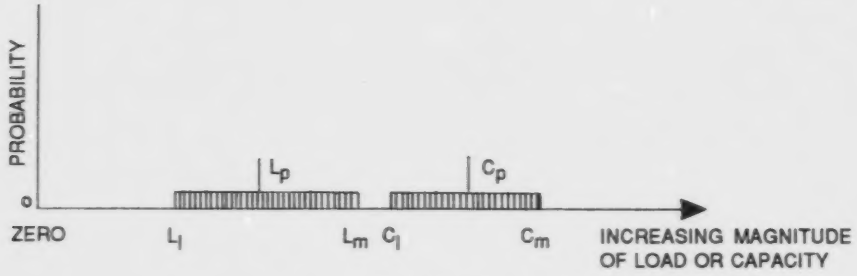


FIGURE 7

unforseen changes. Thus, the real capacity may vary from C_l to C_m.

The engineer then selects a member size such that the capacity (C_l-C_m) is higher (to the right on Figure 7) than the load (L_l-L_m).

We can get a better sense of the problem by reference to Figure 8, where the probability of any value of L or C being the correct one is plotted vertically.

There is a high probability that L_p, the National Building Code value, will indeed be the maximum load. There is a low probability that the load will get as low as L_l or as high as L_m. Intermediate values of load have probabilities represented by a smooth curve such as that shown. Because of their shape, such curves are called bell curves when the causes of variation are random.

Similarly, there is a high probability that the calculated value of the capacity, C_p, will be correct, and lower probabilities that it will be as low as C_l or as high as C_m.

Now the object of the designer is to place C_p far enough to the right of L_p to reduce the *probability* that the load will exceed the strength to an acceptably low value.

For simplicity, we have illustrated capacity and load values as having finite ranges, but we can never guarantee that there will not be extremely high or low values of loads or strength.

Thus, the bell curves really overlap and failure is always, even if only remotely, possible.

The subscripts $_l$ and $_m$ do not really refer to extreme values but to reasonably low and high values. There is a very low probability that strength will be less than C_l or that the load will be more than L_m.

The stronger we make the structure — the further to the right we move (C_l - C_m) — the more it costs. Therefore, it is a matter of balancing economy against the probability of failure.

In practice, the most probable load L_p is multiplied by a *load factor* greater than 1.0 to get L_m the "factored" load. The most probable capacity C_p is then reduced by

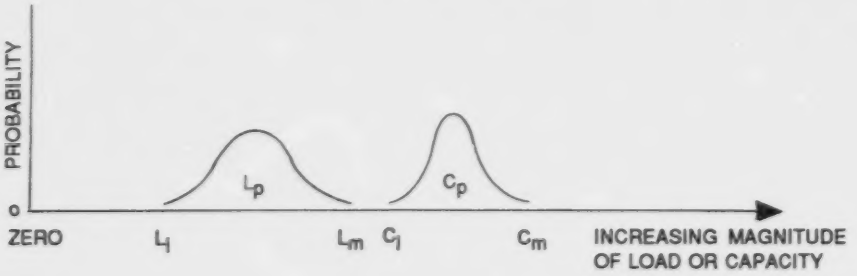


FIGURE 8

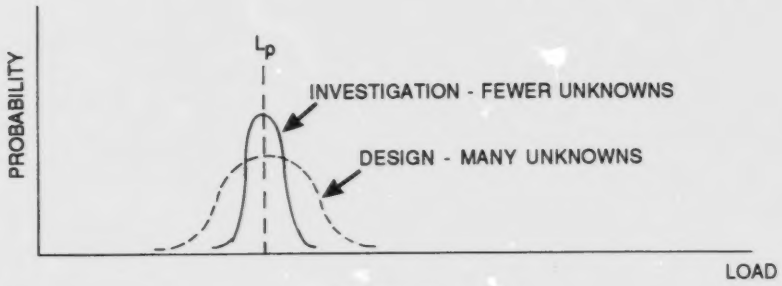


FIGURE 9

multiplying by a *capacity reduction factor* (always less than 1.0) to get C_l or factored capacity.

The structural designer chooses a member such that the C_l is at least as great as L_m, as shown on Figures 7 and 8. The result will be safe unless *simultaneously* an extraordinary high load is applied and an extraordinary low capacity exists. Because suitable safety factors are used, the vast majority of buildings are safe.

Structural Investigation

If a failure has occurred the problem is to determine which of the many possible modes was the cause.

We are not, now, looking at a structure which is to be built in the future. We are considering, instead, a building and its loads, which existed at 10 minutes past nine on the morning of April 23, 1988.

We know the loads fairly accurately. The thickness of concrete has been measured, and the weight of the cars that fell in has been determined with reasonable accuracy. Thus, the probability curve for the load has a relatively-narrow range and looks something like that shown in solid lines in Figure 9.

The strength of the member that failed can then be calculated with respect to each of the possible failure modes. Many of the uncertainties are still present, but the steel material of the specific beam has been tested so that the strength has a probability distribution with a slightly reduced range compared to that usually in effect during the design process.

In our investigation, we are not concerned with factored loads and strengths. The question is quite straightforward: in which failure mode did the actual load reach the actual strength and initiate the collapse? The answer, however, is less straightforward. Let us plot the three possible failure modes (Figure 10):

Failure Mode A — The probable capacity is definitely greater than the probable load. Nevertheless, it is remotely possible that the actual load is much higher than the probable load, and that the actual strength is much less than the probable strength. Thus, it is possible, but unlikely, that failure occurred in this mode.

Failure Mode B — The probable strength is slightly greater than the probable load, but actual strength could be less than actual load. This is a more likely mode of failure.

Failure Mode C — The probable strength is less than the probable load. The actual strength is possibly greater than the actual load, but this is not probable. Of these three possible modes, failure probably occurred in Mode C.

Our conclusion would be that Mode C is probable, but the other two are possible.

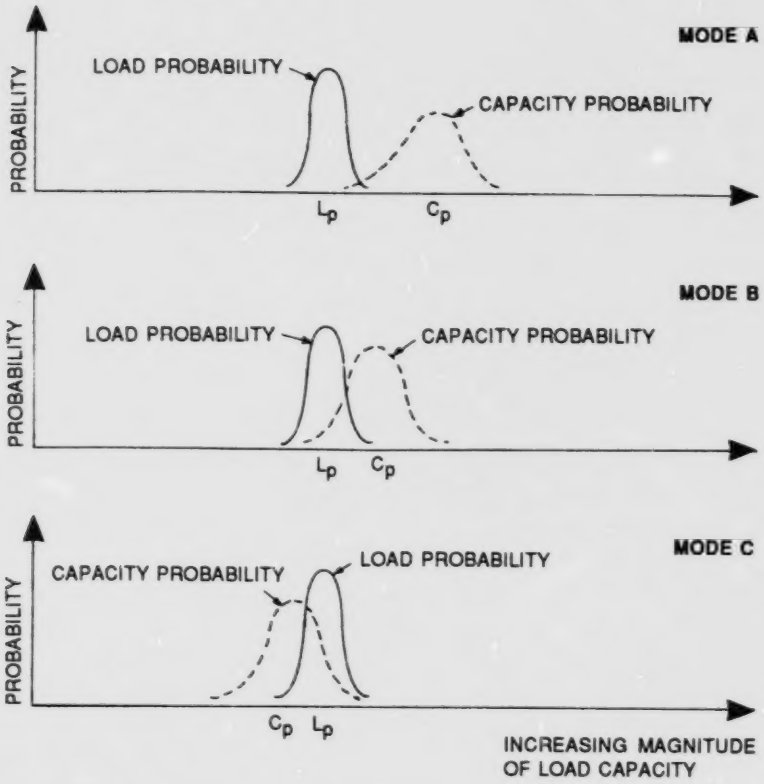

MODE A
LOAD PROBABILITY
CAPACITY PROBABILITY
PROBABILITY
L_p
C_p
MODE B
LOAD PROBABILITY
CAPACITY PROBABILITY
PROBABILITY
L_p
C_p
MODE C
CAPACITY PROBABILITY
LOAD PROBABILITY
PROBABILITY
C_p
L_p
INCREASING MAGNITUDE
OF LOAD CAPACITY

FIGURE 10

CHAPTER 3 — *Station Square Structure*

Roof Deck

To provide a pavement for cars parked on the roof, a three-inch thick wearing surface topped the roof construction. See Figure 11. In the center of the roof was a six-inch-high walkway.

Below the topping was four inches of light styrofoam insulation. Under the walkway this increased to 10 inches.

Below the insulation was the waterproof membrane which was applied to the top of a composite corrugated steel-concrete structural deck with concrete specified to be two and one half inches above the corrugations.

The above-deck structure, except as it affected the loads, was not a factor in the collapse.

Steel Joists

The deck was supported by factory-fabricated steel trusses called OWSJs, or open web steel joists. The joists were spaced six feet and one-and-one-half inches apart. They spanned north-south from beam line to beam line, a distance varying from 35 to 46 feet. Each end of the top chord of each joist sat on and was welded to a beam.

Again, apart from affecting the load on, and the lateral support of, the beams, the joists were not factors in the collapse.

Steel Beams and Columns

Twenty-four-inch deep steel W-section beams spanned east-west and were supported by columns. Column spacing varied from about 30 to 45 feet.

The beams did not span simply from column to column. Each alternate beam extended over its two supporting columns and was cantilevered beyond them by about six feet. Beams AB and CG are cantilevered beams in Figure 12.

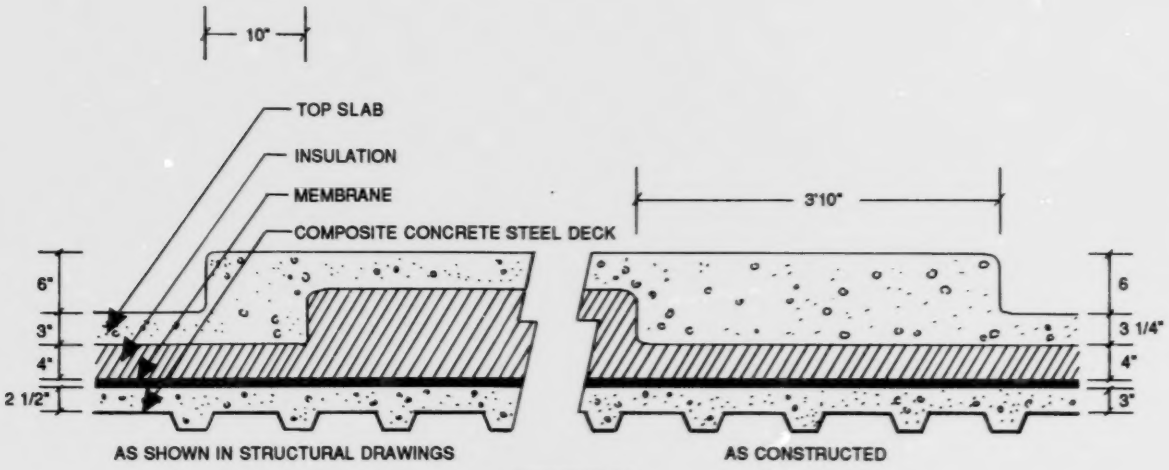


FIGURE 11

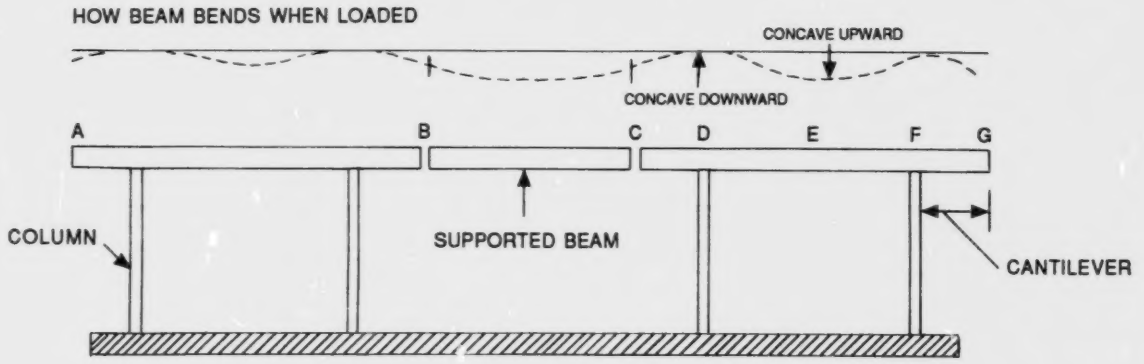


FIGURE 12

Shorter beams such as BC were then supported by the cantilevered ends. This is the Gerber system or cantilever suspended span (CSS) system. It is commonly used for roofs. Correctly designed, it provides a safe structure with greater economy than simple column-to-column spans.

The steel columns were 12 × 12-inch hollow steel sections supported on concrete spread footings. The Inquiry found no evidence that the footings were a factor in the collapse.

Nature of the Collapse

Failure started in a cantilevered beam similar to beam CG and, further, it occurred over a column such as D in Figure 12.

The photographs taken between the onset of failure and the final collapse, some four to five minutes later, clearly identify the initial failure as occurring at this location.

As can be seen in Plate 1, the beam at that time had bent so that the web was almost horizontal and the lower flange and the top of the column had moved to the south.

As the beam moved from a vertical to a horizontal position, the joists and deck sagged and went into tension. They acted as a very shallow "suspension bridge" structure for that vital 4½ minutes, allowing the shoppers and staff to get clear. They then let go and the final collapse occurred.

CHAPTER 4 — *Causes of the Collapse*

Loads

Loads were calculated as accurately as possible.

The live load consisted of the parked cars. A list of these vehicles, their empty weight and the area they occupied gave a close estimate of the actual live load at the time of collapse. (This load was substantially less than the 50 psf design load.)

Dead load was established by measuring the actual material in the deck, the walkway, and the steel structure. It was noted that the thickness of the two concrete slabs was, on the average, slightly greater than that specified, see Figure 11.

The total dead load for the deck generally was estimated at 100 psf, including five psf for hung loads, see Plate 4, page 20 (lighting, sprinklers, signs, etc.) Measurement of the walkway gave a relatively precise load for this element.

With the loads established, the actual moments, shears and forces acting on the beam and the column were calculated, and compared with the corresponding capacities.

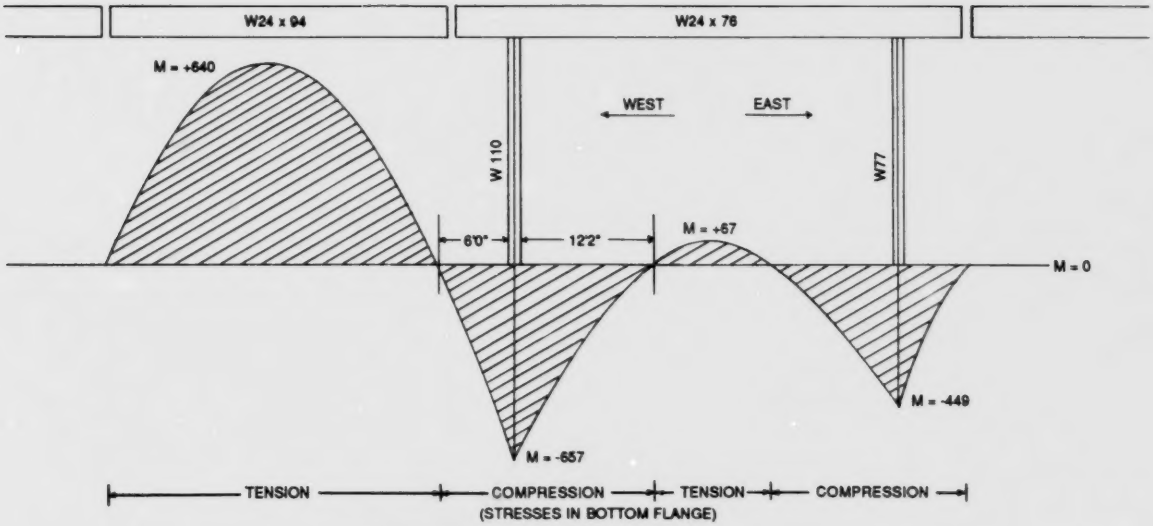
Bending — Caused by Loads

The loads caused bending moments in the beam that varied along its length, and are illustrated in Figure 13.

The maximum moment is 657 Ft.-Kips at column line W110, the location of the failure. Where moment is shown as positive, the beam is bending concave upward, where negative it is concave downward.

Bending — Capacity

The specified steel has a guaranteed strength before it yields, or starts to fail, of 44,000 psi. The actual strength was shown by tests on the failed beam to be slightly higher. The test



MOMENTS CAUSED BY LOADS (IN FT. KIPS)

FIGURE 13

of the lower flange showed a strength of 48,900 psi.

The moment capacity of the beam depends on this strength and the beam's geometry. For a W24 × 76, the probable moment capacity is 815 Ft.-Kips provided the compression flange is laterally supported.

This capacity is reduced if the distance between points of lateral support is greater than 10 or 11 feet.

In Figure 13, the negative (concave downward) bending moment in the W24 × 76 beam extends from the end of the cantilever six feet west of the column on line W110 to a point 12 feet two inches east of that column. This means the bottom flange was in compression over that length, and the question of lateral support against buckling governs the strength.

When the moment changes to positive, the bottom flange goes into tension and it is normally safe to assume the tensile force will hold the flange in line, and act as a restraint against lateral movement.

In this particular beam, this assumption is doubtful because the positive moment and, therefore, the bottom flange tension, is minimal.

Normally, the column would be laterally supported by braces to the open web joists lining up with the column. This occurs at line W77 but no braces were installed at W110. The column itself has some resistance to being pushed aside but is not a positive restraint against lateral movement of the bottom flange.

Determination of the effective unsupported length is not cut and dried. It is at least 12.2 feet. It may be 18.2 feet or even longer. There is a good probability that the effective unsupported length is of the order of 16 feet.

If the unsupported length is 14 feet, the moment capacity is about 707 Ft.-Kips; if 16 feet, about 636 Ft.-Kips.

Note: These values are ($M_r \div 0.9$), an approximation of the probable capacity. For a fuller discussion, see Appendix III *Interim Report on the Cause of the Collapse*.

Also it should be noted that in good structural design, lateral support is usually provided at definite locations and the unsupported length is known.

Plotting the bell curves of moment due to load, and moment capacity for unsupported length = 16 feet, we get Figure 14 which shows that failure in this mode was highly probable.

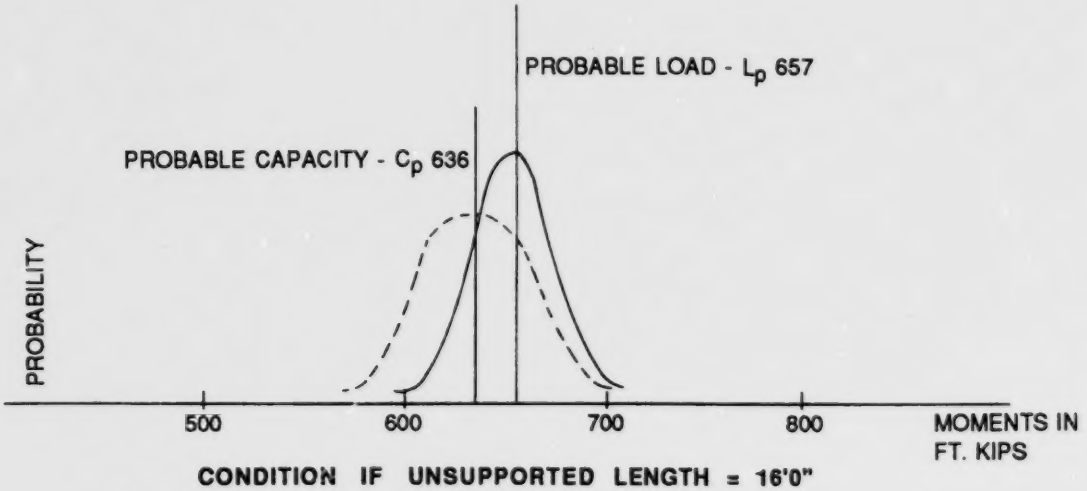


FIGURE 14

Before reaching this conclusion, we considered the impact of the following:

- — reduction of moment to the face of the supporting column;
- — effect of some moment being distributed to the column and the resulting reduction of the 12.2 foot dimension in Figure 13;
- — effect of reduction of section and of stress concentration caused by bolt holes in lower flange;
- — torsional effect of unequal joist span; and,
- — the effect of residual stresses created by the manufacture of wide flange sections.

We decided that there is no precise way of calculating these effects. Some are favourable and some unfavourable to the relative position of capacity compared to load. In our judgment, the probability of this mode of failure would not be appreciably changed by these effects.

Bending failure due to buckling of the unsupported lower compression flange was probably the prime cause of failure.

We note that with the full design live load, together with the actual dead load, the factored moment due to loads would be about 1,028 Ft.-Kips. For this load, even with full lateral support, a larger beam, for example a W24 × 117, or a W30 × 99, is required.

Buckling — Beam-Column

As was discussed above, the strength of the bottom flange of the beam in compression is reduced by its ability to move out sideways to escape the load. It was noted that the strength depended on the "unsupported length" — the length of compression flange that was not prevented from moving sideways.

It is also possible for the column to move out to the side and escape some of its load. When it does this it will, of course, take the bottom flange of the beam with it, and they will both get out from under the load they are supposed to carry. This we refer to as buckling of the beam-column system.

This type of collapse is very dangerous; it occurs suddenly, without warning, and is usually catastrophic in nature. Therefore, in the design process, one does not normally make elaborate calculations to determine the load at which it will occur. Rather, if there is any possibility that it may be the governing condition, one ensures that it will not occur by such steps as inserting braces to prevent the lateral movements.

No braces were provided in the beam-column system that failed.

The photographs taken during the collapse show that the beam-column system did eventually buckle. This does not necessarily mean that it was the initiating process in the collapse.

If the beam failed in bending first, so that the bottom flange of the beam was in the plastic state, its ability to resist sideways movement would be reduced, so that buckling could follow.

In the investigation process, we have to undertake the involved calculations necessary to determine whether buckling was the governing mode of failure.

Essentially, the investigation proceeds as follows:

We can assume the shape into which the structure will deform as it buckles. This can be approximated quite closely and the result is not excessively sensitive to the exact shape.

We can calculate the work necessary to bend the members into that shape, as well as the work done by the loads as they follow the structure when it deforms. For example, as the column bends sideways, its top moves slightly downwards; the loads are lowered, and they do work as they move down.

Work is scientifically defined as the product of a Force times the Distance through which it moves. If the work done by the loads as the structure deforms is equal to or greater than the work necessary to do the deforming, buckling can occur.

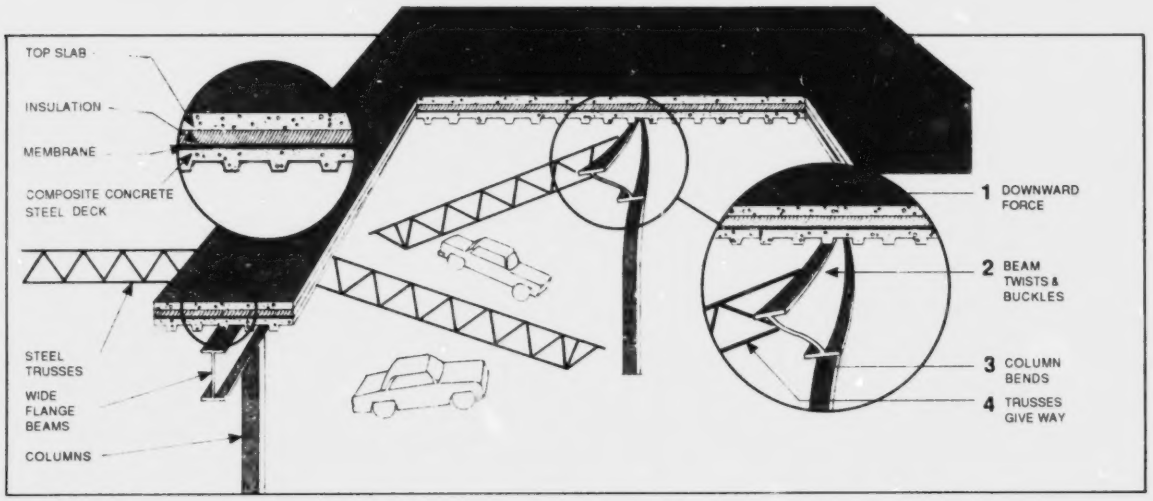


FIGURE 15

The calculation can be done by hand by assuming some mathematically manageable shapes for all the members, or it can be done on the computer.

The Commission had access to four analyses. Those by Sandwell Swan Wooster, by the Workers' Compensation Board, and by the Commission itself suggest that buckling of the beam-column system could have been the primary cause.

The analysis by Jones Kwong Kishi suggests that rotational restraint provided to the top flange of the beam by the open web joist connections could have been sufficient to prevent buckling. This factor was not taken into account in the other analyses.

A study of these analyses shows that there are many small influences that can affect the result. Consequently, we do not feel that these analyses have a high degree of reliability.

We conclude that the instability mode could have been the primary mechanism for the collapse, but that this mode is not as probable as the bending of the beam.

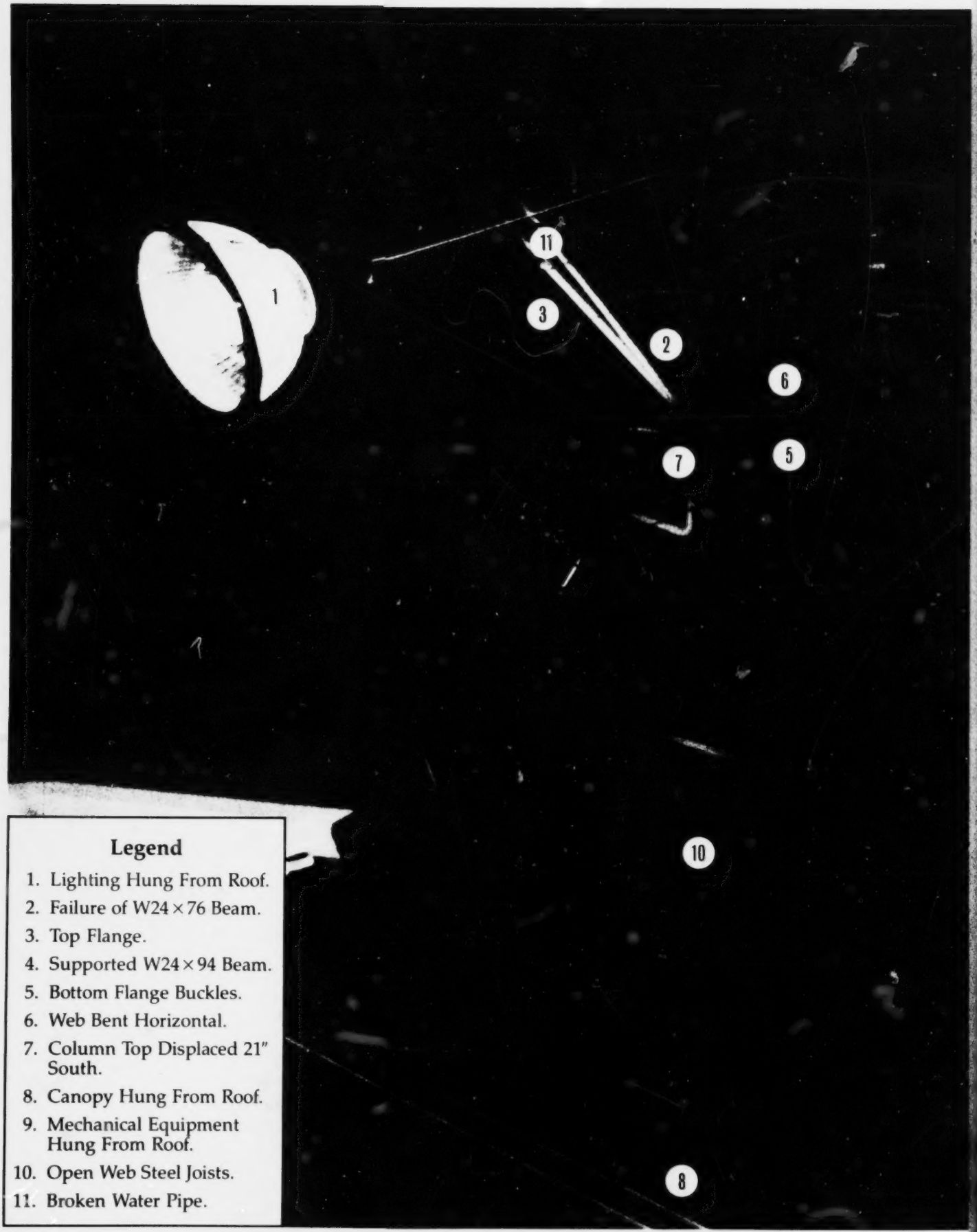
Conclusions

Failure involved two modes.

Mode A — A beam bending failure with buckling of the lower flange. The effective unsupported length of the lower flange is not less than 12 feet and may be 18 feet or more. At 16 feet, the probable moment capacity is less than the probable moment created by the April 23 loads. This is the most probable prime mode of failure.

Mode B — Buckling of the beam-column assembly. Our calculations indicate that this is a less probable prime cause but may have acted together with Mode A.

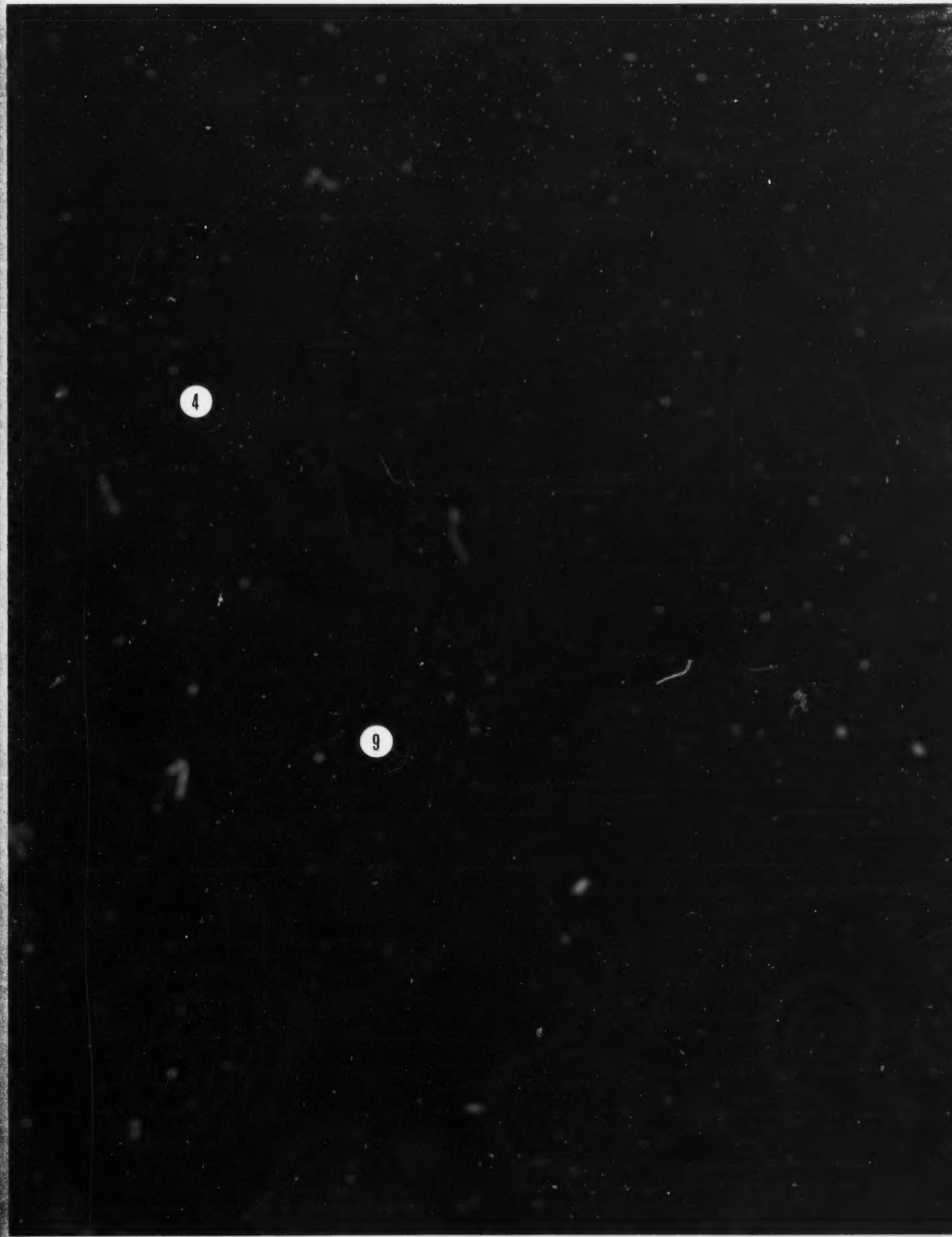
The beam was greatly underdesigned. The beam column assembly lacked essential lateral supports. Failure was inevitable. If not under April 23 loads, then under full service loads.



Legend

1. Lighting Hung From Roof.
2. Failure of W24 × 76 Beam.
3. Top Flange.
4. Supported W24 × 94 Beam.
5. Bottom Flange Buckles.
6. Web Bent Horizontal.
7. Column Top Displaced 21" South.
8. Canopy Hung From Roof.
9. Mechanical Equipment Hung From Roof.
10. Open Web Steel Joists.
11. Broken Water Pipe.

PLATE 4



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CHAPTER 5 — *How the Error Was Made*

Beam Downsized

The beam and column that failed were designed by an experienced engineer, detailed by an experienced detailing firm, fabricated and erected by an experienced steel sub-contractor for a pre-qualified general contractor.

Further, they were independently reviewed by, identified as questionable, and finally given the blessing of a second, independent, experienced structural engineer.

How did the substantial design error get by all these qualified people?

For reasons not fully known, the beam was changed from W24 × 104 to a W24 × 76 — a beam too small even if laterally supported. When doubt was raised, a series of recalculations was made.

In italics below are nine factors involving the design engineer — assumptions, decisions, judgments or miscalculations, each of which reduced the safety margin.

Any one alone would not have resulted in collapse, being partially compensated for by factors of safety. However, taken together, failure was inevitable.

Design Team

Design team was Tamm Tacy and Associates, a structural engineering firm in Vancouver for more than 25 years.

They assumed the live load to be 50 pounds per square foot (psf) as required by the building code. The dead load was calculated to be 80 psf. This included the weight of the steel structure, plus the two-and-one-half inch structural composite concrete slab, an allowance for loads hung from the ceiling, and a two-inch top slab providing the wearing surface.

This top slab was later changed to three inches as shown on the engineer's drawings. This increased the dead load to 80 psf, exclusive of self weight and hung loads.

This is reflected in the notes on the drawings which call for 80 psf, plus the weight of joists and beams, and an allowance for hung loads or a total of 90 to 95 psf.

The open web steel joists were designed by a sub-contractor for this heavier load but the ***beam design was not changed*** by the consulting engineers.

As built, both slab thicknesses were greater than specified and, as a result, the dead load was further increased to about 100 psf. ***There was no provision for this additional concrete.***

No allowance was made for the weight of the walkway because it was detailed on the structural drawings as consisting largely of insulation. In fact, as constructed (see Figure 11), each edge had a three-foot-ten-inch width of solid concrete, increasing the dead load on the beam that failed by about 24,500 pounds.

These three factors, in italics above, increased the dead load moment in the beam by 55 per cent. The dead load moment at the critical column increased from 460 Ft.-Kips for assumed loads to 716 Ft.-Kips for actual loads.

The significance of this increase is obvious as it more than used up the safety factor of 25 per cent added into the calculations as required by the building code.

The design team consisted of the engineer of record, who was a principal of the firm, and of a designer who was a university graduate engineer-in-training with training as a structural engineer. The principal did the preliminary design (including establishing the type of construction, the column layout and the 24-inch depth of beams).

The designer performed most calculations and, assisted by draftspeople, produced the drawings. The drawings were in general checked by the principal who also monitored the job through discussions with the designer. The principal also set design criteria.

Original Calculation

The original design was based on two criteria, both of which had to be satisfied:

— the moment capacity had to exceed the factored moment caused by the assumed loads correctly calculated as 820 Ft.-Kips (i.e. the beam must have the desired strength);

— deflection under dead and live loads together could not exceed 1/240 of the span (i.e. the beam must have the desired stiffness).

The designer concluded that deflection governed the design and a W24 × 104 was chosen. Its moment capacity is 954 Ft.-Kips if fully supported, and exceeds 820 Ft.-Kips for unsupported lengths up to 22 feet.

This size of beam was shown on the structural roof drawing S5.3. It was checked by the principal and issued. Subsequently, a discussion in Tamm Tacy's office about economy led to a review of design criteria, and to a decision to change the deflection criterion to a maximum of 1/360 of the span for live loads only.

Second Calculation

The revised deflection criterion was some 40 per cent less stringent for the live and dead loads assumed. Deflection no longer was considered the governing requirement. The required moment capacity of 820 Ft.-Kips became governing, and a second design was prepared.

The Inquiry was unable to find any record or recollection of the details of this second design.

Nevertheless, subsequent issues of drawing S5.3, including the seventh revision, show the beam ***incorrectly reduced to a W24 × 76***, which has a moment resistance of only 660 Ft.-Kips even if laterally supported.

The procedure used by Tamm Tacy to identify revisions to drawings was to number and describe briefly each revision thereon in a column labelled "Revisions," and to mark the revision in the body of the drawing by a small triangle containing the revision number.

The change in size of the beam was not so identified.

After the initial drawing has been checked, subsequent revisions require a check only of the revisions. An unidentified revision could easily escape notice of the principal.

Third Calculation

After the MSS Group's March, 1988, report identified the beam as deficient, Tamm Tacy did further calculations.

A reduced live load was used to calculate the required moment.

The code recognizes that a live load that may be fully present over a small area is less likely to be fully present over a larger area. It gives a formula for reduction in live load as the loaded area increases. Correct application of the formula results in a minimal reduction to about 49.5 psf.

The area contributing to the moment was incorrectly defined by Tamm Tacy and, as a result, ***the live load was incorrectly assumed to be 43.7 psf***, reducing the required moment from 820 Ft.-Kips to about 770 Ft.-Kips.

They calculated that the W24 × 76 had a capacity of 660 Ft.-Kips, but did not note that this was only if the unbraced length was less than about 10 feet.

Fourth Calculation

When beams are continuous over columns, the bending moment in the beam is highest at the column center line and may be appreciably less at the edge of the column.

In concrete structures, the column and the beam are fully integrated or composite, and the lower, edge, moment value is the maximum applied to the beam section alone. This lower value is used in reinforced concrete beam design.

In steel construction with bolted connections, the beam may act less integrally with the column. Standard practice in cantilevered beam steel design is to use the moment at the center line, although it is recognized that this theoretical moment may not occur.

Tamm Tacy, as a fourth calculation, ***reduced the moment to that at the face of the column cap plate***, to a value of 682 Ft.-Kips or only three per cent more than the 660 Ft.-Kips capacity. Although this calculation was done, Tamm Tacy went ahead with design of a strengthening bracket for the beam, intending to make up this small deficiency.

Mill Certificate

Tamm Tacy obtained "mill certificates" from the steel sub-contractor, Empire Iron Works Ltd., and noted that the lowest value for yield strength was more than 55,000 pounds per square inch (psi). The 660 Ft.-Kips capacity is based on the 44,000 psi guaranteed minimum yield strength of the specified steel.

The mill test information was given to The MSS Group.

In its written submission to the Inquiry dated June 15, 1988, MSS said the mill test reports indicated a minimum yield strength 25 per cent higher than the strength used in their design check, and on this basis concluded that the W24 × 76 beam was satisfactory.

In an earlier letter to Wesbild Enterprises Ltd., MSS enclosed a report on another beam that used a mill certificate showing that the yield strength was 54 Kips per square inch instead of the 44 ksi normally used in design, and concluded that the moment capacity could be increased proportionally.

The assumption made by the engineers — that the "guaranteed" minimum yield strength of a beam can be upgraded to the value given by one value or the lowest of several values given by mill tests — is unsupported for two reasons:

1. The mill test is made on a sample cut from the web of a W shape. The web always has a higher test strength than the flange. For example, tests done on the beam after failure showed yield strengths of

top flange	47.2 ksi
web	53.4 ksi
bottom flange	48.9 ksi

   The moment capacity is dependent on the flange strengths, not on the web strength.

2. Variations in chemical composition and in temperature during the rolling of the shape result in variable strengths. A single or a small number of tests will not give a statistically valid indication of actual strength. The normal bell curve distribution must be examined to determine the most probable actual value.

Tamm Tacy concurred with an MSS Group letter, dated April 7, 1988, which reported that the mill tests made the W24 × 76 satisfactory.

Lateral Support

A W24 × 76 has a moment capacity of 660 Ft.-Kips, provided the compression flange is supported laterally at intervals not exceeding about 10 feet. At greater intervals, the capacity is reduced.

Tamm Tacy's drawing S4 stipulated lateral support at some columns, but not at the column that collapsed.

When open web steel joist shop drawings were circulated, Tamm Tacy returned a set with a note requiring additional supports. When told that an extra cost would be involved, it cancelled the note.

Evidence received by the Inquiry indicated that ***Tamm Tacy did not study the reduced capacity of the W24 × 76 caused by lack of lateral support.***

Lateral support can be provided by braces from the open web steel joists or, in many circumstances, by adding web stiffeners to the beam. Tamm Tacy's principal stated at the Inquiry that he believed that the decision whether or not web stiffeners were required lay with the steel detailer.

The detailers in turn said they did not make provision for web stiffeners unless specifically requested by the engineers in charge because they did not have enough information on loads available to make that determination.

Lack of lateral support reduces not only the beam's bending capacity but also the load-carrying capacity of the beam column assembly. ***This was not studied.***

With lateral support at the column, the W24 × 104 originally chosen by Tamm Tacy probably would have never been detected as deficient, and would have been:

— adequate both in bending and stability for the April 23, 1988, loads; and

— adequate for all service loads except either the weight of the extra concrete or the extra weight of walkway both of which were not known to them at the time.

The W24 × 104 would be eight per cent less than the code requirement if loaded with actual dead load and the 50 psf code live load. It would have survived all normal service conditions, again provided that it was laterally supported.

It should be noted that Tamm Tacy had no information that the actual concrete thicknesses exceeded those specified; and that the extra walkway width consisted of concrete.

CHAPTER 6 — *The Contractors*

Steel Detailer

Steel was detailed on shop drawings prepared by Dowco Consultants Ltd., an independent firm supplying a detailing service to steel fabricators and contractors.

Its managers have technical training and wide experience in structural steel detailing. It has no professional engineers.

Dowco's function included selection of standard beam connections — that is connections having a capacity equal to the reaction from the supported beam when it is fully loaded in bending by a uniform load.

The connections were not factors in the collapse.

Dowco did not consider it was its function to decide whether web stiffeners were required, because it did not have the necessary load information or knowledge of the overall structural concept.

Structural Steel Sub-Contractor

Empire Iron Works Ltd. was the structural steel sub-contractor. Empire did no engineering calculations and prepared no drawings.

Open web steel joists were supplied by Great West Steel Industries Ltd. as a sub-contract to Empire. GWS designed these joists and supplied shop and erection drawings for them. They did no erection.

All other detailing was sublet by Empire to Dowco.

The contract drawings do not show details of connections.

The contract required that shop drawings for work designed by the fabricator bear the stamp and signature of a Professional Engineer.

Except for joist drawings, Empire involved no engineers in the preparation of shop drawing and provided no stamped or sealed drawings because it felt that the shop drawing is a fabrication aid and not an engineering drawing.

There was a difference in interpretation of the contract but the Inquiry heard no evidence to indicate that this had any bearing on the collapse.

General Contractor

Amako Construction Ltd., the general contractors, allocated one superintendent specifically to the Save-On-Foods building.

He was backed up by a general superintendent and a project manager, both of whom were involved in other projects. All construction was by sub-contractors.

There were two possible interpretations of revision 6 on the architect's drawings in which the rooftop walkway was widened from five feet six inches to 11 feet six inches.

On the architect's drawing, the instructions read: "Extend sidewalk three feet zero inches into parking stalls to form tire stop."

Amako agreed that the architect's instructions could mean either adding concrete or adding insulation. Amako added concrete with the resulting substantial increase in load to the beam that failed. Amako said this was a design matter and that it was up to the the architect and engineers to ensure that their instructions leave no room for error.

CHAPTER 7 — *Independent Review*

Review by MSS Group

The MSS Group was engaged by Save-On-Foods because of concern about deflection of the beams.

In italics below are seven factors, decisions, assumptions, judgments or errors made by MSS.

Each of these factors resulted in a reduced safety factor.

As part of its review of the main roof beams, MSS identified the beam that collapsed as suspect and the moment at the column as critical.

Load Assumptions

MSS had the contract drawings and access to the almost complete building. Its engineers assumed the live load to be 50 psf as required by code and as called up on Tamm Tacy's drawing S5.3. Dead load was assumed to be 80 psf total.

Drawing S5.3 gave the load superimposed on the joists as 80 psf. ***MSS did not allow for the weight of the joists and of the beams.*** This load amounted to about six psf.

Drawing S5.3 also required that provision be made for hung loads as shown on Plate 4. These include the normal electrical and mechanical equipment and the bulkheads, canopies and signs that are characteristic of the supermarket business. ***MSS made no allowance for hung loads.*** The magnitude of such loads can vary. An allowance of five to 10 psf is common. MSS, should have allowed not only for the existing hung loads, but also for possible future changes.

The walkway had been completed at the time of the MSS review. As shown on architectural drawing A5.5, it was originally five feet six and one half inches wide and consisted largely of extra thickness of insulation.

The extra insulation had no significant weight. The only extra weight was in the 10-inch-wide edge of the walkway shown on structural drawing S5.5.

The architect's drawing was subsequently revised and the walkway was enlarged to 11 feet six and a half inches.

The contractor could have conformed by either using more insulation and maintaining the 10-inch-wide concrete edge; or keeping the same insulation and widening the 10 inches of concrete to three feet 10 inches at either side. He chose the latter.

The resulting weight of the walkway tributary to the beam became some 24,500 pounds. The effect of this load is to increase the moment by 113 Ft.-Kips or 18.5 per cent. ***The weight of the walkway was ignored in MSS calculations.*** It would have required a questioning mind to uncover this factor.

By ignoring the hung loads, the weight of the joists and the walkway, MSS underestimated the moment caused by dead load by 46 per cent.

Further, MSS did not know, and could not reasonably be expected to know, that the two roof slabs were constructed thicker than specified, resulting in higher actual moment and increasing the above percentage to 55 per cent.

First Calculation

MSS, using the underestimated total dead load of 80 psf and working stress procedures, calculated the moment at the column to be 43 per cent greater than the capacity of the W24 × 76.

Second Calculation

The second calculation was for the same loads, but using limit states design. The moment was 25 per cent greater than the beam capacity.

Third Calculation

Next, a reduced live load was considered as provided for by the code and as discussed above. ***The tributary area was incorrectly assumed to have a length equal to the span, plus both cantilevers.*** For this area, a live load of 41 psf resulted. The correct reduction would be to 49.5 psf. With the incorrectly reduced live load, the overstress was reduced to 15 per cent.

Fourth Calculation

A fourth calculation then reduced the moment to the edge of the column. The overstress became eight per cent. As pointed out earlier, ***such a reduction is not normal practice in steel construction.***

MSS then designed a strengthening bracket to increase the capacity to the calculated moment but the remedial work was not carried out following receipt of a mill certificate.

Mill Certificate

After obtaining information from Tamm Tacy that the lowest mill certificate yield strength of the specified 44,000 psi yield strength steel was over 55,000 psi, MSS wrote its April 7, 1988, letter concluding that the beam was "at least 26 per cent stronger than the specified strengths and . . . . satisfactory to resist the design loadings imposed."

This conclusion is unwarranted for the reasons discussed in chapter 5.

Lateral Support

As indicated earlier, the lack of lateral support reduces not only the beam's bending capacity, but also the the load-carrying capacity of the beam-column assembly.

The Inquiry has ***no evidence that MSS considered the need for lateral support.***

In fact, testimony by the firm's principal showed that MSS was on the site five times and failed to see that lateral support of the bottom flange was not provided though this was readily visible. All MSS calculations were based on developing the full, supported, moment capacity.

CHAPTER 8 — *Conclusion*

The task of the Commission in Part I was to determine the cause of the collapse and to find out why this tragedy occurred.

Failure involved at least two modes.

Mode A — A beam bending failure with buckling of the lower flange. The effective unsupported length of the lower flange is not less than 12 feet and may be 18 feet or more. At 16 feet, the probable moment capacity is less than the probable moment created by the April 23 loads. This is the most probable prime mode of failure.

Mode B — Buckling of the beam-column assembly. Our calculations indicate that this is a less probable prime cause but, together with Mode A, may have resulted in simultaneous failure in both modes.

The beam was greatly underdesigned. The beam-column assembly lacked essential lateral supports.

Failure was inevitable — if not under April 23 loads, then under full service loads.

The event and the reason why have been clearly documented on the preceding pages.

The investigation carried out by the Commission raised several tributary issues that will concern people who are involved in the construction industry.

The need for greater care by all parties in communicating special and unusual requirements to suppliers and designers is of vital importance on all building projects.

The four-and-one-half minute delay prior to the final collapse averted a greater tragedy with the resultant loss of life and additional serious injury.

The people involved in the next disaster, whatever that may be, may not lose only their cars.

PART II

PREVENTION

CHAPTER 9 — *Overview*

The second part of the Commission's terms of reference instruct me . . . "to ascertain and recommend measures that could serve to prevent similar tragedies in the future."

The first part of this report described the respective roles of the various parties involved in the construction of the Station Square Development. These interests ranged from the land owner through to the municipality, the architects, engineering consultants, the general contractor and the subtrades.

The evidence received by the Commission about the roof collapse, and in relation to the organization and division of responsibilities among these various parties, is confined largely to this specific project.

Nevertheless, this isolated occurrence has raised broader issues of public policy having to do with public safety in the construction of buildings in the province.

The practices and procedures followed in the construction industry vary substantially from project to project, depending on the organizational structure adopted by the owner, the design of the project, its size and intended use, and the professionals and others who are involved in the project as it proceeds to completion.

The Commission has received a number of submissions from individuals and organizations involved in the industry, concerning the organizational and professional responsibility aspects of construction. A number of themes have emerged as a result of the near-catastrophe at Station Square and the subsequent policy debate.

First and most obvious, the role, training, and accreditation of the professional engineers in the construction industry must be critically examined.

Second, the responsibilities of municipalities in the construction process have been raised repeatedly and these, too, require examination.

Third, the owner plays a pivotal role in shaping a project and in influencing its pace and the chain of responsibilities among the professionals, contractors and others involved. While owners are difficult to identify as a cohesive group, their role ought to be reviewed.

Finally, subtle issues are raised by the relationships among the owners, professionals and contractors, and the resulting standards of work on construction projects. This process requires a general review involving major interested parties.

These issues are addressed in the following chapters of this report.

CHAPTER 10 — *Project Organization*

Construction projects in the province take on a variety of shapes and sizes from ordinary residential buildings to large highrise and industrial structures.

The organizational framework adopted for each of these projects varies according to their size and complexity. The issues raised by this Inquiry are focused primarily on major developments. This chapter of the report is intended to describe, in general terms, allocations of responsibility that are typical for larger projects requiring specialized structural engineering involvement.

Owners and Project Managers

In its simplest form, a project is conceived by a real estate developer, or owner, who acquires parcels of contiguous land with the intention of building a residential, commercial or industrial structure on them.

Others, in the public sector, are conceived by federal, provincial or municipal governments for institutional purposes, such as schools, hospitals and other public purposes.

In urban areas, existing structures often are demolished before construction can proceed. If the project involves a change of land use, rezoning applications are made to the appropriate municipal authorities, sometimes with the assistance of architects or other consultants.

At that point, owners typically adopt one of two approaches to organizing the construction of the project.

Traditionally, they appoint a "prime consultant," normally an architect, who is responsible for overall design; employing structural, mechanical, geotechnical, electrical and other engineering consultants; calling construction tenders; recommending a general contractor to the owner; qualifying sub-contractors; coordinating drawings and other documentation, including changes to them; approving payments on the owner's behalf to all parties; and providing for general site supervision himself or through a consultant.

In recent years there has been a trend away from this traditional approach and towards greater involvement of the owner directly in the overall organizational administration of the project.

Under this structure, the owner employs a project manager who is actively involved as the owner's representative in appointing not only the architect, but also in carrying out many of the erstwhile functions of the architect.

Thus, the project manager takes responsibility directly for employing and overseeing the engineering consultants, tendering building contracts, approving payments by the owner to all consultants and contractors, and ensuring that general supervision and inspection on site are carried out.

These two organizational models are not intended to be an exhaustive description of the structures employed today by owners in the construction industry in the province. Rather, they illustrate two general approaches. Individual projects may adopt a combination of the two, with architects assuming many of the responsibilities of project managers.

Engineering Consultants

Major construction projects invariably require engineering expertise. Structural engineers are engaged to design the structure of the building in consultation with the project architect. Others are similarly retained to

carry out soils tests for foundations, to design heating, ventilation, utilities and other mechanical features, electrical systems, and so on, depending on the nature of the building and its intended use. As explained above, the engineering consultants can be retained by the architect, or directly by the owner or the project manager on behalf of the owner.

Contractors and Sub-Trades

The contractor is responsible for constructing the building according to the design and specifications determined by the architect and engineers. Normally, the contractor is engaged by the owner following a tendering process, usually for a fixed price. It is common practice for contractors, in turn, to sub-contract many or all of the components of a building to sub-trades and suppliers: structural steel, concrete, mechanical elements, insulation and internal fixtures, to name a few.

Municipal Authorities

The role played by municipalities in the construction process is regulatory, being concerned largely with ensuring that local zoning requirements are met and that structures conform to applicable building codes. A National Building Code has been developed for Canada.

British Columbia, through the Ministry of Municipal Affairs, Recreation and Culture, has modified the national code for the province. The provincial building code applies throughout the province, except for the City of Vancouver which has adopted the provincial code, with some modification, in its building by-law.

(To simplify this report, references to "municipalities" are intended to include regional districts and the City of Vancouver, where appropriate.)

Owners who propose to construct buildings are required, through these means, to conform to structural specifications set out in the building code. In general, Part 4 of the code sets out structural standards for public, institutional and all relatively large buildings, while Part 9 is a counterpart for residential and small commercial structures.

These, then, are the main participants in a typical large construction project.

In succeeding chapters of this report, the functions and responsibilities of many of them are examined with a view toward ensuring public safety in the construction process and avoiding structural collapse in the future.

CHAPTER 11 — *Municipal Responsibilities*

It was explained in the previous chapter that, in the construction industry, municipalities play a largely regulatory role through zoning and the application of the building code of the province.

Questions raised in this Inquiry about the responsibilities of municipal governments have arisen, in some part at least, as a result of misconceptions by the public about the function of the building department. This confusion is understandable in light of the division between Parts 4 (large buildings) and 9 (small buildings) of the code, and misconceptions about the assigned functions of building inspectors.

This report will try to clear away these misunderstandings and to explain the intended role of municipalities and municipal officials under the code and related by-laws.

The provincial building code applies, under the *Municipal Act*, to structures erected in all municipalities and regional districts in the province. The City of Vancouver, which has its own provincial *Charter* separate from the *Municipal Act*, has adopted the code through by-law.

In most cases, municipalities have supplemented the code requirements with by-laws dealing with such things as the transportation of buildings, fees, use of street areas and ancillary matters.

Building permits serve as the primary link between the code and individual construction projects. In general, a project may not proceed unless authorized by a building permit issued by the municipality. Permits are issued by building inspectors who are municipal employees. Municipal by-laws set fees for building permits.

For design purposes, the code creates two categories.

The procedures for the issuance of building permits for Part 4 and Part 9 structures vary, and the distinction between them is important for this report and for municipal responsibilities.

The code requires that all Part 4 buildings be designed by an architect or an engineer. In practice the building inspectors rely on the professionals to make sure that the Part 4 buildings comply with the code.

For the smaller Part 9 structures, building inspectors play a far more active role when building permits are issued, during construction, and on completion.

Part 4 Buildings

For Part 4 building permits, construction documents, including structural drawings, are submitted to the municipality, together with "letters of assurance" given by the project owner generally identifying a "prime consultant" for the project. The design calculations themselves are not normally submitted.

The prime consultant is referred to as the "professional designer" and must be an architect or structural engineer.

Further "letters of assurance" are provided by the designer, certifying that he will provide required site inspections during construction and for other aspects of the building. Eventually, one or more building permits are issued, authorizing construction.

Normally the consulting engineer who has taken responsibility for the preparation of the structural drawings places his seal on them.

Although some municipalities check the drawings, albeit superficially, others do not. For Part 4 buildings, municipalities tend to rely on the expertise of the design

professionals in ensuring the integrity of the detailed design of buildings.

During the course of construction, building inspectors carry out random site inspections, but, here again, they are not expected to carry out detailed reviews.

Contrary to public perception, the building inspectors do not assume responsibility for the structural adequacy of Part 4 buildings.

On completion of the project, some municipalities issue interim and final occupancy permits, authorizing use of the building, on receipt of another letter of assurance from the designer certifying compliance with the code. Others decline to issue them, apparently out of concern that they might attract legal liability for any faults in the structures.

The Station Square Development collapse has focused attention on municipal responsibilities in Part 4 buildings.

Some participants in the Commission hearings suggested that it be mandatory for all drawings and calculations to be certified and sealed by two independent professional engineers.

Others recommended that the practice in certain European countries, where the state carries out detailed checks of all major building structural designs, be adopted in the province. Under our system, these checks would be carried out by municipal or other local authorities.

While such measures might help to reduce the probability of error in the design of Part 4 structures and might clarify lines of authority, the associated costs would exceed the benefits.

For the few structures that pose potential risk to the public, requiring *all* designs to be checked would lead to unnecessary duplication and impose undue costs on municipalities and the construction industry.

However, I believe that current procedures should be tightened to reduce the risk of structural collapse. The severe potential consequence of error associated with structural collapse requires new procedures for monitoring designs and calculations.

Therefore, I recommend that:

1. The provincial building code should be amended to require structural design monitoring procedures for Part 4 buildings, as follows:
   - — Structural calculations prepared by structural engineers of record for Part 4 buildings should be submitted to municipalities, along with structural drawings, when application is made for building permits.
   - — A random sample of 10 per cent of the designs and calculations for all proposed Part 4 buildings in each municipality should be selected for detailed review.
   - — In addition, where municipal building officials believe that imposed loads, extraordinary use or unique features of other Part 4 buildings so warrant, the design of those buildings should be submitted to a panel of the Association of Professional Engineers of the Province of British Columbia (APEBC). The panel should review these designs and, where the panel considers it desirable, designs should be selected for detailed review.
   - — The reviews should be carried out on behalf of the municipality by consulting engineers specified by the APEBC.
   - — The costs of the reviews should be funded by a special levy imposed by all municipalities in the province on building permits for all structures governed by Part 4 of the provincial building code.
   - — Funds generated by the levy should be paid into a pooled interest-bearing trust fund to be administered by the APEBC.

— The trust fund should be audited annually and the amount of the levy should be reviewed periodically in relation to the cost of administering reviews.

— The trust funds should be used only to pay the fees of consulting engineers who carry out detailed reviews of selected Part 4 building designs.

— The *Engineers Act* should be amended to accommodate these procedures, if necessary.

— The engineer of record who originally prepared a design under review should not, by reason only of the review, be relieved of responsibility for the design.

These proposals should ensure that Part 4 building structural designs are subjected to audit by qualified professional engineers.

I am satisfied that the prospect of random reviews, in itself, should enhance the integrity of structural design services for Part 4 buildings throughout the province. The additional selection of unusual structures for review should add further protection to the public.

The cost of this process, which I estimate at roughly five per cent of building permit fees in the province, will be modest in relation to the gains it offers in improved public safety.

Certain administrative shortcomings associated with Part 4 buildings have come to light during the course of this Inquiry.

Earlier in this chapter I referred to the "letters of assurance" required by municipalities in connection with building permit applications and the completion of construction.

The forms used for these letters vary from municipality to municipality, causing uncertainty and confusion in the construction industry.

I consider that the public interest would be served if these letters were the same throughout the province and systematically implemented through the appropriate channels.

Therefore, I recommend that:

2. The building code should be amended to incorporate standard letters of assurance to be used throughout the province in connection with buildings governed by Part 4 of the code. These letters should be required of the owner, and of architectural, structural, mechanical, plumbing and electrical services which are provided directly or indirectly by professionals, prime consultants and sub-consultants.

The role of building inspectors in Part 4 projects requires clarification. Under current arrangements, practice varies from municipality to municipality. In some, inspectors regularly visit construction sites while in others, they do not.

A major thrust of Part 4 of the building code is to delegate to professional engineers and architects the responsibility for ensuring compliance with its requirements.

If both design professionals and building inspectors check documents and make site inspections, some confusion about responsibilities is engendered.

With the review process recommended here, it should be unnecessary for building inspectors to be involved with Part 4 structures. There would be little for them to add in light of the limited resources available and training they receive. Their time could be more efficiently deployed in other areas.

Accordingly, I recommend that:

3. For Part 4 buildings, the design professionals should have clear and prime responsibility for building code

compliance, both at the design stage and during construction. Building inspectors should have a secondary responsibility, limited to ensuring that the appropriate design professionals have accepted prime responsibility. Municipal building by-laws should be amended accordingly, and such amendments should eliminate any requirements for building inspectors to carry out field inspections for Part 4 buildings.

Part 9 Buildings

Buildings governed by Part 9 of the code are smaller and tend to be structurally less complex than those subject to Part 4. For the most part, they include residential and commercial buildings of three storeys or less.

Arrangements for the administration of building permits for Part 9 structures reflect this difference. Professional designers are not required to prepare the drawings, nor are "letters of assurance" normally required from owners or professionals to certify compliance with the code. Although architects and engineers are often engaged to design these buildings, they are not generally required by the code to be involved.

Municipal building officials, conversely, play a larger role. They carry out detailed checks of drawings to ensure that all code requirements are complied with before building permits are issued.

Building inspectors conduct site inspections throughout the course of construction, again to ensure compliance. Thus, many of the functions that are delegated to professionals for Part 4 structures are assumed by municipalities for Part 9 buildings.

There appears to be no reason to change the code procedures as they relate to Part 9 buildings, so I have no specific recommendations to make in this regard. If, as I have recommended, involvement of municipal officials in Part 4 buildings is reduced, this staff should be able to concentrate their efforts on those under Part 9 and on other responsibilities.

By-Laws

During the course of the Inquiry, it has become apparent that building by-laws of many municipalities are out of date with regard to the current building code. These by-laws refer incorrectly to the 1980 code, whereas a new version was approved in 1985.

This obsolescence has no legal effect, because the code, once approved, applies throughout the province even without ratification by municipalities. Nevertheless, it can potentially create confusion in the minds of the public, particularly less sophisticated builders.

Consequently, I recommend that:

4. The Minister of Municipal Affairs require all municipal building by-laws to refer to the current edition of the building code.

CHAPTER 12 — *Responsibilities of the Owners*

Owners who erect buildings on their own land have legal responsibilities to ensure they are structurally safe. Under general principles of liability in the province, land occupiers must take reasonable care towards the safety of persons who come onto the land. Thus, if a person suffers injury on the land because of a faulty structure, an owner could be held responsible.

As well, if an owner fails to ensure that provisions of the building code as set forth in the administrative requirements for use with the national building code are properly followed in the erection of a structure, he could face prosecution.

Concern has been raised at this Inquiry about practices that have developed in the construction industry over recent years, and their effect on public safety. Three contentions have been raised.

First, there has been a trend toward "fast-tracking" of projects. That is, in an effort to minimize the duration of the project and hasten the time at which financial returns on land and building investments are realized, owners are accelerating the pace of construction.

"Fast-tracking" occurs when construction begins before design is complete. It is argued that this practice leaves insufficient time for building design professionals to properly discharge their responsibilities, could lead to error, and, in turn, jeopardize public safety.

Ideally, the design of a project should be complete before construction starts. Thirty years ago, many projects approached this ideal.

The degree of fast-tracking can involve a small number of minor design changes during construction, to a myriad of changes, both minor and major.

At one extreme, construction may get under way with only the general layouts and detailed structural drawings needed to place footings. Construction of the supporting structure may continue with little more than the necessary structural drawings.

To prepare these drawings, the structural engineer uses only tentative information about the loads the structure will carry. Gradually, other drawings are prepared, based on more complete information.

If the owner is concurrently leasing the space to prospective tenants when construction is well along, changes may continue regularly to suit the requirements of these tenants. Each change affects one or more, maybe many, participants in the project — both in the design and the construction.

The interests affected by the changes must be identified and their feedback obtained. In turn, their responses might prompt further changes.

In a large commercial project, the flood of changes requires skill, care and energy to coordinate and direct. Even with clear lines of responsibility, the number of changes and the number of players affected makes the possibility of errors substantial.

Second, owners and their representatives are increasingly calling for tenders from architects and engineers involved in Part 4 buildings. In the past, fees for these services normally were based on a fraction of the value of the project, but with tendering, relatively intense competition has driven fee levels down, and this has raised questions about the quality of professional services in this environment.

Third, the emerging role of project managers as the representatives of the owners in developments in some instances leads to a blurring of responsibility between the various

parties — the owner, the architect, the engineers, and the contractors. This phenomenon is referred to as "fragmentation," and can occur both in design and construction.

Under the building code, the owner has full responsibility for ensuring that the design, construction and occupancy of the building complies with its requirements. To discharge this duty, he has three options: exercise this responsibility himself if he is competent to do so; delegate full responsibility to a single master builder or prime consultant; or delegate parts to various competent people.

Lines of responsibility under the first two options above are relatively clear. The third, however, results in divided responsibility. Even if the divisions are clearly delineated and fully understood, the various parts lack a leader or controller, and chances of mistakes abound.

On the design side, ideally, a prime consultant coordinates and directs all the design disciplines. The owner then directs the prime consultant only.

In such an arrangement, the owner clearly delegates to the prime consultant (usually an architect) the responsibility for all design and for administering the contract: that is, for seeing that the design is, in general, carried out. Under this model, the owner speaks only to the prime consultant.

When the owner directly selects, appoints and pays the other members of the design team, the role and responsibilities of the architect become blurred.

The owner, far from communicating only with his prime consultant, frequently involves himself in meetings of the design professionals and even in construction site meetings.

The fragmentation is further aggravated by the attendance of the tenant at such meetings. At these meetings, both the owner and the tenant may direct the various design or construction team members.

On the construction side, the general contractor undertakes to produce a building conforming to the drawings and other contract documents. The general contractor in the past carried out a substantial part of the work with his own resources and personnel whom he knew and controlled.

Today, increasingly the general contractor has only a small controlling office with a superintendent or other senior staff, secretarial helpers and a few labourers. Almost all work is sub-contracted.

The Station Square project is an example — the roof structure alone involved these sub-contractors:

— a steel sub-contractor who fabricated and erected structural steel and erected joists;

— a joist sub-contractor who designed and supplied joists;

— a steel detailing sub-contractor;

— a concrete supplier;

— a steel deck supplier;

— a steel deck erector;

— a reinforcing steel sub-contractor; and,

— a concrete finishing sub-contractor.

To these can be added roofing, insulation, electrical and mechanical sub-contractors. The work of each of these sub-contractors may impinge on the work of one or more of the others. Coordination and direction are essential, complex and difficult.

In the absence of a prime consultant, the responsibility for coordinating and directing design and construction is blurred.

The three practices — fast-tracking, bidding for professional services and fragmentation as discussed above — cause me a great deal of concern. The role played by owners in the construction process places them in a key position to influence the quality of construction and, thus, the safety of the public in the province.

Nevertheless, I have concluded that it would be inappropriate to attempt to regulate the

conduct of the owners beyond requiring compliance with the building code.

Owners, as such, are not governed by standards of ethical practice as are architects, engineers and other professionals. Neither are they a clearly defined group in the community. While some land developers have been in business for many years and are represented by an industry association in the province, others become involved only briefly in isolated "one-shot" projects.

To meet the concerns described above, I consider it more appropriate to focus on the professional standards of engineers and architects, and construction industry procedures, and this is reflected in my recommendations in the following chapters of this report.

CHAPTER 13 — *Responsibilities of the Architects*

The professionals who are most directly involved in the structural elements of construction are the architects and the structural engineers, although other engineering disciplines are called on as well. The regulation of these professions has clear and important implications for public safety in construction standards.

The practice of architecture in British Columbia is administered by the Architectural Institute of British Columbia under the *Architects Act*. Like other statutory professional organizations, architects are self-governed by an elected council.

Architects often serve as professional designers and prime consultants under the building code. However, their responsibilities normally do not extend to design of structural elements in Part 4 buildings, the major focus of this Inquiry.

Evidence received by the Commission has not disclosed any serious shortcomings in the professional organization of the architectural profession, within the terms of the Commission's mandate. Nevertheless, the evidence suggests that architects have difficulty in complying with the use and occupancy (Part 3) provisions of the building code, an issue that I address in chapter 14. Further, architects should be encouraged to participate in the development of the construction practices and procedures manual, recommended in chapter 15 (recommendations 11 and 12).

CHAPTER 14 — *Responsibilities of the Engineers*

The crux of this Inquiry lies with the engineering profession. Had it not been for miscalculations by structural engineers, a section of the roof of the Save-On-Foods store would not have collapsed. Had it not been for the collapse, this Inquiry would not have been struck.

Professional Organization

Engineers are organized along lines similar to those for other self-governing professional bodies in the province, under the *Engineers Act* (administered by the Ministry of Advanced Education and Job Training) and associated by-laws.

The profession is governed by the Association of Professional Engineers of the Province of British Columbia, through an elected council and executive. The association maintains offices in Vancouver with full-time professional and clerical staff.

In general terms, the act and by-laws provide for the organization and administration of the association, qualification and registration of individuals as members of the association and disciplinary proceedings against members. The by-laws include a code of ethics. A member may be disciplined following conviction of certain offences, contravention of the act, by-laws or code of ethics, or *"incompetence, negligence or unprofessional conduct"* (emphasis added).

Penalty under the disciplinary proceedings can take the form of a reprimand, the imposition of conditions on membership, suspension of membership or revocation of membership.

It is understood that the association is conducting an investigation into the conduct of the structural engineers involved in the Station Square project. Therefore, the Commission will not make any recommendations in this regard.

Before an individual qualifies for membership in the association, typically he is required to graduate with a degree in applied science from a recognized university. He then serves for at least two years as an engineer-in-training under a qualified professional engineer and this experience must be satisfactory to the association.

As well, before becoming qualified, he is required to pass an examination, set by the association, that is concerned largely with ethical and legal aspects of the profession. Successful completion of this examination entitles the applicant to membership in the association, to use the title "Professional Engineer" (or the abbreviation P.Eng.) and, as such, to sign and seal professional documents.

A newly-qualified member is registered with the association in one of 18 engineering disciplines, including structural, which has featured prominently in this Inquiry. Others, including civil, mechanical, electrical and geotechnical, are also involved extensively in the construction industry.

Normally, the discipline selected is based on the nature of the applicant's university degree and his experience as an engineer-in-training. The association does not examine applicants with accredited university degrees in their disciplines before qualifying them for membership. Nor does the association monitor the nature of engineering services members actually perform once registered.

A member registered in one discipline may actually work in other disciplines. However, members are required, by the code of ethics, to restrict themselves to work for which they are "qualified."

Engineers are among the largest of the self-governing professional bodies in the province. Currently, roughly 11,000 are registered as members — 1,000 serve as

engineers-in-training and 300 are licensed non-residents, bringing the total to approximately 12,300 under the aegis of the association.

Of this total, 713 are registered as structural engineers. For the reasons explained above, the number of association members who actually practice in this discipline is unknown. An estimated 132 structural consulting engineering firms operate in the province.

The number of consulting firms in all engineering disciplines totals approximately 300, employing roughly 2,500 engineers and a further 2,500 non-professionals. Active engineers not engaged in consulting are employed by business corporations, governments, and so on.

All of these statistics were provided to the Commission by the association.

Registration of Firms

Under section 18 of the British Columbia *Engineers Act*, a corporation offering engineering services is required to have on its active staff one or more registered members or licensees who directly supervise and take responsibility for the practice of professional engineering undertaken by the corporation. Through these means, the requirements of the act cannot be circumvented through incorporation.

This feature is common in legislation aimed at professions that allow members to practise in corporations. However, some jurisdictions — notably Alberta, Ontario, New Brunswick and Newfoundland, and at least one U.S. jurisdiction, the State of Washington — carry regulation of firms a step further.

Individuals engaged in providing engineering services are required, as in British Columbia, to be registered. However, *firms* offering these services to the public are also required to be registered separately.

The registration of the firm may be revoked where the firm has engaged in unethical, unprofessional or incompetent practice, in addition to disciplinary proceedings that might be taken against individual engineers in the firm. Registration requirements of this nature serve two purposes.

First, they recognize today's business realities. It is common practice for corporations and similar commercial structures to serve as vehicles for engineers for tax planning, third party liability and other purposes.

Second, and more germane to this Inquiry, regulatory measures such as these can serve to strengthen standards within the engineering profession. Facing the prospect of decertification of a firm as a whole, the individual members within the organization will have a strong incentive to ensure that through internal checks high standards of service are provided.

Therefore, I recommend that:

5. Companies, partnerships, firms and other associations that provide professional engineering services to the public should be required under the *Engineers Act* to be registered in addition to registration of individual engineers. Such registration should apply to all engineering disciplines.
6. Such companies, partnerships, firms and associations should face deregistration for unethical, unprofessional or incompetent practice. Such measures should be in addition to disciplinary proceedings taken against individual members.

I understand that measures such as these have been seriously considered recently by the association. I urge all concerned to proceed to this end without delay.

Registration of Structural Engineers

This chapter earlier described registration requirements of professional engineers under the *Engineers Act* and by-laws, including the

review by the association of the experience of engineers-in-training. The examination that must be passed before registration is not intended to test an applicant's substantive knowledge of his discipline.

For most engineering disciplines, these procedures appear to be satisfactory. While, to some degree at least, inept performance in them can impinge on public safety, the potential damage from errors is usually relatively small.

Structural engineers, on the other hand, stand on a different footing. As we have seen with the Station Square roof collapse (and other projects such as the catastrophic 1958 failure of the Second Narrows Bridge which claimed 23 lives), a structural error can lead to serious tragedy. Due to the particularly high potential cost of structural error, a reform of registration requirements is paramount.

I recommend that:

7. The by-laws of the Association of Professional Engineers of the Province of British Columbia should be amended to require that before registration as a structural engineer, the competency of the structural engineer be tested by a special written examination or additional supervised training, or both. Only engineers who qualify in this way should be eligible to professionally certify drawings and calculations of Part 4 structures under the building code.

   The building code should be amended accordingly. These qualifications should not be required of currently registered engineers unless the association determines that their competency is questionable.

8. Structural engineers should be required to carry a specified minimum of professional liability insurance coverage.

Professional Fees

Earlier, this report referred to increased fee competition among engineers in the province. This trend has resulted, in part, from rising competition for work and changes in federal competition policy.

Bidding for the provision of professional services may result in the provision of those services at a lower rate, in the provision of fewer services at the same rate, or in some combination of the two.

Engineering services associated with the construction of a building such as Station Square ideally consist of:

- preliminary studies of the structural form to determine the optimum layout, choice of materials and so forth;
- the performance of precise calculations to establish member sizes and details;
- the preparation of drawings which convey this information in unmistakable terms to the contractor;
- the careful checking of all these calculations and drawings by an independent individual within the organization;
- the preparation of specifications defining the materials and procedures to be used during construction;
- the transfer of all the necessary information to the contractors who will, perhaps, make further calculations and prepare shop drawings for the fabrication of items such as steel beams, joists, and their connections, precast concrete, etc;
- the careful checking of these shop drawings to ensure that the details are correct, or, at the least, that they conform to the overall concept;
- inspection in the field to ascertain that the construction proceeds in general accordance with the design drawings and specifications;
- the revision of the drawings as the work proceeds, especially in "fast-track" operations, to accommodate changes

called for by the architect and owner; and,

— the dissemination of this new information, again in unequivocal terms, among all those involved.

If the charge-out rate for engineering personnel is estimated conservatively at $50 an hour, the number of man-hours budgeted for by those structural engineering firms which entered proposals on the $5,400,000 Save-On-Foods building varied from 260 to 640. There were three bids below 410 man-hours, two between 410 and 580, and three above 580.

The conclusion is inescapable that all companies did not intend to provide the services listed above with the same degree of diligence. Indeed, the "causes of the collapse" show that failure to carry out some of the steps with unhurried precision led to this event, which, but for an incredible stroke of luck, would have had consequences too ghastly to contemplate.

The situation, as revealed by these events, simply cannot be tolerated. We must take advantage of this narrow escape from catastrophe to ensure that public safety is not again put at such risk.

Some solution must be found to make certain that the professional engineers can and will devote the necessary man-hours to a project to perform in full the services listed above.

Two general approaches are possible:

— Pressure the engineers to provide complete services so that they will be unable to submit unrealistic fee proposals; and,

— Pressure the owner of the building to provide sufficient compensation to permit the engineer to do the work properly.

Other recommendations in this report are designed to achieve the first. I turn now to the second approach.

The association maintains a "schedule of recommended fees" for members who practice as consultants. Compliance with this schedule is voluntary only and the association does not attempt to discipline members who deviate from it, out of concern for the "price fixing" provisions of the federal *Competition Act* and its predecessor the *Combines Investigation Act*. That is, the schedule is not enforced.

Prior to 1976, federal competition policy did not apply to the provision of services, such as engineering work, but was targeted largely on providers of goods. That year, the legislation was broadened to include services, prompting the association's position with regard to fees since then.

I understand that the federal authorities are reluctant to take action against restrictions in price competition for professional services where such practices are clearly authorized by provincial legislation. Courts have confirmed this limitation.

As a result, the question becomes whether it would be in the public interest for the British Columbia Legislature to amend the *Engineers Act*, authorizing an enforceable fee schedule for engineers.

I have concluded that an enforceable fee schedule for engineers who offer their services to the public could assist in maintaining acceptable professional standards. Therefore, I recommend that:

9. The *Engineers Act* should be amended to permit the APEBC to establish and enforce a schedule of minimum fees, to be approved by the Lieutenant-Governor-in-Council.

Design Standards

The building code serves as the central means by which the government influences construction practices in the province.

Evidence received by the Commission points to wide discrepancies in the quality of designs submitted to municipalities in support of building permit applications.

Fire and safety requirements were singled out as recurring examples.

Many design professionals are not familiar with the Use and Occupancy (Part 3) provisions of the code. Drawings submitted are often incomplete in this respect and too much reliance is placed on municipal staff. I consider this to be a serious shortcoming.

These procedures require tightening, to establish uniformity of approach among designers and to ease the burden on the municipal officials who are responsible for receiving drawings and, as I have recommended earlier, design calculations.

I recommend that:

10. Provincial standards of practice should be established for the preparation and submission of all building design drawings and calculations governed by Parts 2, 3, 4, 5 and 8 of the building code.

With respect to fire and safety, several witnesses at the public hearings expressed the view that the four exits at the front of the Save-On-Foods store were insufficient. I have concluded that, considering that only four-and-one-half minutes were required to evacuate 1,000 people from the building when the roof verged on collapse, the exits were adequate.

However, during the course of the Inquiry, I viewed fire exits in a number of other stores and have become aware of potential hazards. In many instances, stock piled in passageways and stacked high above shelving units could hamper evacuation under earthquake or other emergency conditions.

While I have no specific recommendation to make about this problem, I am drawing attention to it.

Concluding Note on Engineers

At the outset of this chapter, the engineering profession was identified as central to this Inquiry. Recommendations in this and the previous chapter are intended to improve its effectiveness in ensuring public safety.

These recommendations do not propose a complete overhaul of the profession and the way it is governed. That is unnecessary. What is needed, however, is a concerted effort by the profession to stiffen its standards and re-tool some of its procedures.

CHAPTER 15 — *Construction Practices, Procedures*

Two themes have recurred in this Inquiry, in relation to construction practices: "fast-tracking" and the fragmentation or blurring of responsibilities on and off site.

Unless proper provision is made for the additional engineering costs for the extra time and effort associated with "fast-tracking", tight scheduling can seriously undermine the ability of professionals to properly discharge their functions on a project. This problem is further aggravated when important features of design are changed during the course of construction.

It should be emphasized that "fast-tracking" in itself is not objectionable provided that proper provision is made for the extra time for design and coordination.

Fragmentation has emerged as an issue, as owners and contractors have replaced traditional allocations of work and reporting lines with a variety of organizational structures. As explained earlier, project managers have been inserted into organizations as the representatives of the owners.

To an increasing degree, prime contractors sub-contract work and materials to the point where they often employ only supervisory personnel on site. Under "design-build" sub-contracts, components (such as structural steel) are designed and fabricated off-site under engineering supervision and assembled by the sub-contractor. Lines of communication such as between the sub-contractor and structural engineers on site often lack consistency and are unsystematic.

Clearly, the informal and piecemeal approach to construction practices in the province that has emerged in recent years is ripe for review. Like other industries, the construction sector has evolved over the centuries with changes in technology, new materials, advances in applied science, architectural styles and so on.

We can expect change to continue in response to these and other influences. However, it is critical that the rules governing construction keep pace with evolving building practices and curb trends that threaten public safety.

In this respect, I am convinced that the major participants in the construction industry — owners, municipalities, architects, engineers, contractors and sub-contractors — could benefit substantially from a uniform set of construction guidelines. A close parallel to this approach can be found in the standard form contract documentation that is commonly used in the construction industry.

I recommend that:

11. A manual of recommended construction practices and procedures should be developed for the province.
12. The issues addressed by the manual should include:
    - respective responsibilities of owners, architects, engineers, contractors, suppliers, municipalities and building inspectors, and communication among such parties;
    - designation of a prime consultant on projects;
    - coordination of design and construction, and changes to work on site; and,
    - certification of drawings and calculations by project engineers and the engineers of supplies where appropriate.

Procedures for implementing this recommendation, in consultation with the

construction industry, are set out in chapter 17.

Such a manual, if properly conceived and carefully formulated, should serve as useful "rules of the game" for construction in the province. It would eliminate the cloud of uncertainty currently surrounding construction management and organization. Further, it would reinforce professional standards and buttress other recommendations in this report that are designed to enhance them.

With those in place, and agreed to by all major interests, it would be far easier for the professionals to resist pressures in individual projects that could tend to undermine their standards of service.

CHAPTER 16 — *Steel Industry Recommendations*

Manuals and other publications produced by steel industry organizations are valuable references for structural engineers. In the course of research done by my advisors, it became apparent that, good as these references are, safety probably would be enhanced by a number of modifications.

I am aware that I am directed to make recommendations to government on matters of public policy and that these organizations are not regulated by public agencies. Nevertheless, I consider this report an appropriate vehicle for drawing attention to these manuals.

Action to be taken on these recommendations, as a result, should take the form of requests by government to the appropriate organizations.

Gerber System Guide

The Gerber system, or cantilever suspended span (CSS), is relatively simple where loads are uniform and spans are relatively equal.

Where live loads vary, as with parked cars, or spans vary, comprehensive analyses are required. Because the effective unsupported length of the lower flange changes with the shape of the bending moment curve, the "worst case" load condition is not easily determined, particularly if the ratio of the cantilever to span is greater than about 10 per cent.

The Canadian Institute of Steel Construction (CISC) provides useful information on design of steel. It publishes, for example:

— *Single Storey Building Design Aid* which includes sample calculations and drawings for CSS;

— *Cantilever Suspended Span* software for computers, together with a "Users Manual;" and,

— Course notes for the CISC Short Course on the design of Single Storey Buildings which includes an extensive study of the Gerber system.

Many structural engineers do not have, or are not familiar with, these detailed publications. All, however, are familiar with the CISC *Handbook of Steel Construction*, considered to be the "Bible" on steel design in Canada.

I recommend that:

> 13. The Canadian Institute of Steel Construction should consider adding to the *Handbook of Steel Construction* a Guide to the Gerber System or CSS Design.

This guide should draw attention to the sensitivity of the moment calculations to variable loads and spans and to cantilever length. It should be a guide to:

— Calculating unsupported flange length;

— Criteria for design of braces;

— Continuity created by fixing ends of joists; and,

— Detailing of stiffeners.

CSS Computer Program

The CISC computer program, *Cantilever Suspended Span*, assumes that the point of inflection is equivalent to a lateral support. If the positive moment is small, this may be a questionable and dangerous assumption.

I recommend that:

> 14. The CISC should review the *Cantilever Suspended Span* program and make changes if necessary to provide safe answers where positive moments are small or minimal.

Beam-Column Stability

Design complexities occur not only in beam design but also in beam-column stability. The applicable standard is CAN3 S16.1-M78 *Steel Structures for Buildings — Limit States Design*, prepared by the Canadian Standards Association. This code is published in the *Handbook of Steel Construction*, where it is followed by a CISC commentary on the code.

Section 15.2 of the Standard states: "Beams and columns shall be restrained against rotation about their longitudinal axes at points of support."

There is, however, no commentary on section 15.2. The corresponding American standard draws more specific attention to the care required where beams sit on top of columns. The CISC should add to its commentary a similar commentary or one such as:

"A beam or girder resting on top of columns should have the bottom flange stayed laterally to ensure development of the necessary negative bending moment capacity."

In addition, section 19 bears on this matter and commentary 19.2 should have added:

"When a beam or girder is continuous over a column, the stability of the column girder assembly should be assessed. Lateral support should be provided to the bottom flange of the beam, or the top of the column, or both."

I recommend that:

15. The Canadian Institute of Steel Construction should consider adding to its commentary a provision for the special stability considerations where beams sit on top of columns.

Load Factors

The National Building Code requires that the load factor for dead loads be 1.25, recognizing that, in general, dead loads can be relatively accurately predicted.

However, if the dead load consists of a thin slab, construction tolerances may be critical. For example, if a two-and-one-half-inch thick slab is specified but three inches of concrete is actually placed, the dead load is increased by 20 per cent, largely using up the load factor.

As concrete is poured on a steel deck, deflections occur in the deck, joists and beams. If the top of the slab is required to be a true plane, then extra concrete will be placed to compensate for the deflection with resulting extra thickness and extra dead load.

I recommend that:

16. The Canadian Standards Association should consider adding to *Steel Structures for Buildings — Limit States Design*, a reminder to the structural designer that deflections can cause ponding not only of water, but also of concrete, and that the resulting additional loads should be included in calculating loads. The Canadian Sheet Steel Building Institute should consider the effect of deflection on the composite deck weights published in the catalogue for its members.

Composite Slabs for Parking

The Canadian Sheet Steel Building Institute and the publications of its member manufacturers carry warnings that composite decks do not apply to "repeated impact loads," vibrating loads, or "moving concentrated loads" unless specially studied.

The institute recognizes that parking garages can be built with composite deck provided their *Guidelines for Hi-Bond Steel Floor: Application to Parking Garage Construction* (See Appendix V), are followed. Because of reservations about misuse, the guidelines are available only on request. They require a heavier deck than that used at Station Square.

I recommend that:

17. The Canadian Sheet Steel Building Institute and its members should consider adding to their published design manuals and load tables their reservations about, and guidelines for, use of composite decks for parking structures.

These suggestions, if adopted, would help to reduce the risk of building failure in future by providing more thorough guidance to structural engineers. The three organizations involved, the Canadian Institute of Steel Construction, the Canadian Standards Association, and the Canadian Sheet Steel Building Institute, should be made aware of them.

CHAPTER 17 — *Implementation*

The recommendations formulated in this part will involve or implicate at least two provincial ministries — Municipal Affairs, Recreation and Culture; and Advanced Education and Job Training — as well as municipalities and the full range of interests in the construction industry. Some proposals will involve only a few, while others, by their nature, will require input from many.

Moreover, it is essential that implementation of these recommendations, if acceptable to government, be carried out expeditiously. Because of the variety of interests involved, delays will be inevitable unless a relatively small and efficient working group is mobilized to implement the proposals.

To achieve this objective, I recommend that:

18. The Minister of Municipal Affairs, Recreation and Culture should appoint a task force, consisting of three people who are knowledgeable about the construction industry, to oversee implementation of the recommendations in this report. The chairman and at least one other member should be appointed from the private sector. The task force should be given six months following its appointment to complete its assignment.

Implementation of the recommendations in chapter 15, aimed at the development of a construction practices and procedures manual, should involve a full spectrum of interests in the construction industry. If these rules are to be respected and followed in this sector, it is essential that all concerned have an opportunity to participate in their formulation.

Therefore, I recommend that:

19. To implement the recommendation for a construction practices and procedures manual, the task force should consult with representatives of owners, architects, engineers, contractors, suppliers, municipalities and building inspectors.

CHAPTER 18 — *Conclusion*

At the outset of this report, I explained that the paramount consideration in shaping my recommendations is public safety and that, ultimately, responsibility for public safety rests with government. The recommendations in Part II are aimed at assisting public authorities to this end.

The review of the organization and function of the engineering profession figures prominently in my recommendations. Engineers, particularly those in the structural discipline, occupy a special place and carry onerous responsibilities in the construction industry. It is essential that their training, certification and standards of performance are thorough and up-to-date. My recommendations are intended to provide direction in this regard.

The review process proposed in chapter 11 should provide incentives for carefully prepared structural designs. Registration of engineering firms should lead to tighter quality controls within engineering offices. Province-wide standards of design practices should serve to increase the quality of service while easing the burden on municipal building officials. A more formal approach to professional fees, if properly structured, has the potential to help enhance professional standards.

The responsibilities of the municipal level of government, as land use regulator and administrator of the building code, were examined. Municipalities are well placed to act as a clearing house for the design reviews. Beyond that, other recommendations are aimed at cleaning up administrative anomalies and more clearly defining the role of building inspectors in relation to the design professionals.

A considerable amount of concern was raised in this Inquiry over current construction practices and project organization. This debate features the role of owners in construction, but also involves virtually the entire industry. My proposed construction practices and procedures manual should help to define lines of responsibility and accountability on projects by laying out rules understood by all involved.

My recommendations concerning manuals and other publications that are used by structural engineers in carrying out design work, if acted upon, could improve their usefulness.

Finally, I hope that the measures I recommend to implement this report will be carefully considered. Too often reports such as this languish and are forgotten, not owing necessarily to the merit of their proposals but because of simple inertia.

Elsewhere in this report, it was explained that applied science is not equipped to guarantee that a structure will not collapse. The objective of the engineer is to limit risk to tolerable levels at reasonable cost.

Likewise, neither can a Commission such as this nor government guarantee public safety. However, once deficiencies in organization and administration are identified — this time by a near catastrophe — action must be taken to reduce the risk to people.

APPENDIX I — *Commissioner's Terms of Reference*

PROVINCE OF BRITISH COLUMBIA

ORDER OF THE LIEUTENANT GOVERNOR IN COUNCIL

Order in Council No. 830 Approved and Ordered MAY -6 1988

I hereby certify that the following is a true copy of a Minute of the Honourable the Executive Council of the Province of British Columbia approved by His Honour the Administrator.

Order-in-Council Custodian

Lieutenant Governor
Administrator

Executive Council Chambers, Victoria MAY - 4 1988

Administrator

On the recommendation of the undersigned, the Lieutenant Governor, by and with the advice and consent of the Executive Council, orders that

1. A commission be issued under Part 2 of the *Inquiry Act* appointing Dan J. Cloakey as a commissioner to inquire into and, on or before August 31, 1988, to report on any and all the circumstances surrounding, leading to or having any causal connection with the collapse of a portion of the Station Square Development in Burnaby on April 23, 1988 and to ascertain and recommend measures that could serve to prevent similar tragedies in the future.

2. The remuneration and allowances of witnesses who are required for the purposes of the commission, in respect of mileage and maintenance and other incidental and necessary expenses, including travelling expenses, be on the same scale as provided in the Rules of the Supreme Court of British Columbia.

3. Consent is given to the commissioner appointing clerks, stenographers and research assistants as he considers necessary for conducting the inquiry and to pay them at the rate or salary that is equivalent to the rate or salary paid to employees in similar positions in the public service and the commissioner may appoint or retain counsel and other consultants or advisors that he considers appropriate and the commissioner may establish rates, fees, living and travelling expenses to be paid to those positions.

4. Subject to appropriation, approval is given to government to pay all expenses incurred by the commissioner in the inquiry and which are considered necessary by the commissioner for the proper carrying out of his duties.

5. Remuneration of the commissioner is set at $300 for each day of involvement in the enquiry. Living and travelling expenses be paid to the commissioner equivalent to the rates paid to Group III managerial employees in accordance with Treasury Board Order 88, and that clerks, stenographers and research assistants and similar employees be paid equivalent to the rates paid to Group II employees in accordance with the said Orders, for each day during which they are engaged in the performance of the powers and duties of the commission.

Minister of Municipal Affairs

Presiding Member of the Executive Council

(This part is for the records of the Office of Legislative Counsel, and is not part of the Order.)

Authority under which Order is made:

Act and section:- Inquiry Act, sections 8 and 17

Other (specify):-

Examined by: Clifford E. Watt
(Attorney General examiner)

April 28, 1988

948 /88/jb

APPENDIX II — *Commissioner's Oath of Office*

I, DANIEL J. CLOSKEY

swear that I will truly and faithfully execute the powers and trusts vested in my by His Honour the Lieutenant Governor in Council, under the Inquiry Act, according to the best of my knowledge and judgement.

SO HELP ME GOD.

Daniel J. Closkey

SWORN and subscribed by the said DANIEL J. CLOSKEY at Burnaby, British Columbia, this 11th day of May, A.D. 1988, before me -

Judge and of the County Court of Westminster

APPENDIX III — *Commissioner Inquiry*
Station Square Development
Burnaby, B.C.
Dan J. Closkey, Commissioner

JUNE 1988

AN INTERIM REPORT ON THE CAUSE OF THE COLLAPSE

Prepared by

Dr. Noel Nathan, P.Eng.
and
C. Peter Jones, P.Eng.

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1.0 INTRODUCTION

On April 23, 1988, a portion of the roof-top parking deck over a new Save-On-Foods store in the Station Square Development in Burnaby, B.C., collapsed shortly after the store opening ceremonies.

The provincial cabinet appointed Mr. Dan J. Closkey as sole Commissioner under the *Inquiry Act* to:

1. report on any and all circumstances having any causal connection with the collapse;
2. recommend measures which could serve to prevent similar tragedies.

This interim report deals with the first of these tasks only.

It was prepared when the Commission had heard only a small part of the expected testimony or briefs, and it is based on the information available to the authors.

Further activities may reveal new evidence and may require revisions of this report.

On the architect's drawings, the collapsed area is within the grid line S466.83 to S533.66, and W77.3 to W152.17.

This area is shown on the model.

The authors, based on the information available, believe that a major cause of the collapse is evident and that the beam that collapsed was:

1. too small; and
2. lacked necessary lateral support.

This report was jointly prepared by C. Peter Jones, P. Eng., Engineering Coordinator, and Dr. Noel Nathan, P. Eng., Engineering Advisor, respectively, to the Commissioner.

Short biographies of the authors are in Appendix B.

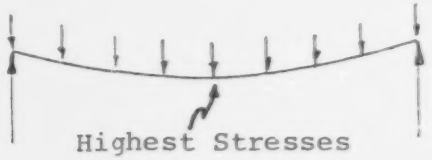
2.0 THE STRUCTURE

2.1 General Concepts

Two concepts will be needed later:

1. Consider a beam resting on two supports carrying some load distributed along its length. It sags in a single curve, concave upwards, and the stresses are highest in the middle.

   If it breaks, it will break in the middle. The ends are sloping down towards the middle.



   Now suppose one grasps the ends and turns them until they are level again. This will raise the centre, and reduce the stresses there.

   However, there is now a section at each end which is bent so that it is concave downwards, while the middle is still concave upwards.

   There are now high stresses at the ends as well, and if the beam fails, it will probably do so at the ends.

Nevertheless, the beam can carry a bigger load in the second condition, and the sag or deflections are smaller. Alternately, a smaller beam can be used to carry the same load.

2. Consider a rod carrying an axial load (along the length of the rod) as a column.

   It can fail in two entirely different and independent ways:

   (a) If it is short and thick, it can be loaded until the material is crushed, and if made of steel, it will suddenly begin to bulge and decrease in length; or

   (b) If it is long and thin, it may suddenly spring out sideways and bow under the load.

   This is buckling, and may occur when the stresses are still quite low.

One of these two modes of failure, "material failure" (or crushing) and "instability failure" (or buckling) may be possible with respect to any member (beam or column) or part of a member which is in compression (subject to a force which is pushing rather than pulling).

Whenever a member or part of a member can escape a compression force by moving out to the side, the possibility of buckling or instability failure exists.

2.2 Structural Form of this Roof

The structure is basically a single-storey column and beam steel building. It uses a cantilevered or "Gerber" system, based on the concept no. 1 discussed above.

It consists of columns, which rise from bases that are essentially fixed, like flagpoles. These columns are spaced about 30 to 45 feet apart in both directions.

Beams are then placed over the tops of two columns, overhanging or cantilevered beyond them by about six feet. (See AB and CG in Figure 1.)

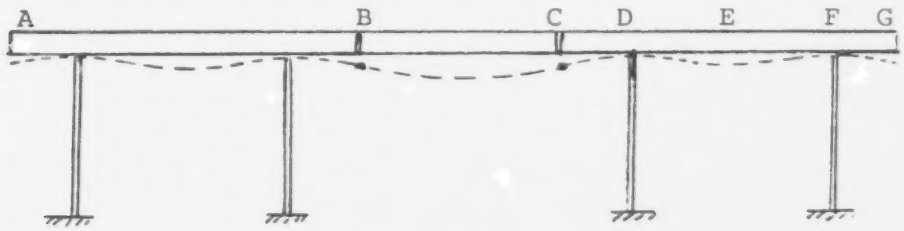


Figure 1

A shorter length of beam BC is then supported between the two overhanging ends as shown.

The beam CG is now in the condition described in the second part of concept 1 above: it is bent so that it is concave downwards over the columns at D and F, and concave upwards at E.

At mid span, it is less severely stressed than if it simply spanned between D and F, but, on the other hand, it does carry significant stresses at D and F as well as at E.

Nevertheless, a smaller beam can be used than would be required to span simply between D and F.

The beam BC is carried simply between its ends, but, since its span is shorter, it can also be a smaller beam.

In the Station Square structure, lines of beams were formed in this way, spanning in the East-West direction, about 40 to 45 feet apart.

The "Gerber" system is a commonly used and an economical system. Correctly designed and built, it safely supports many buildings.

Between these lines of beams, prefabricated steel trusses, or "open web steel joists", span in the North-South direction, about six feet and one and one half inches apart.

They are made of specially-formed steel sections, and their top chords rest on and are welded to the top flanges of the beams.

Corrugated sheet metal decking spans across the six-foot gaps between these, and is welded to them.

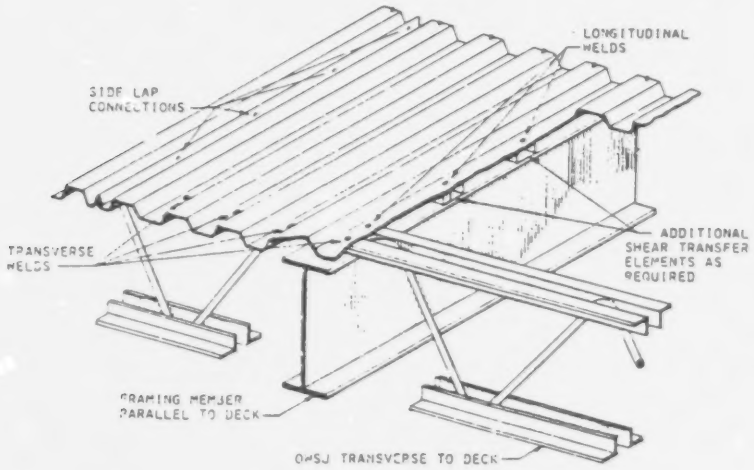
Concrete is then placed on the deck, filling the corrugations and covering them to a depth of about three inches.

This concrete contains a light welded wire mesh and is intended to act together with the metal deck to form a composite system.

On top of this concrete, a waterproof membrane is placed, covered by four inches of styrofoam insulation.

This, in turn, is covered by another three inches of concrete with a light welded wire mesh to act as a wearing surface.

These latter elements are not designed to participate in the structural action of the deck, which is limited to the corrugated metal sheet and the first layer of concrete.


LONGITUDINAL WELDS
SIDE LAP CONNECTIONS
TRANSVERSE WELDS
ADDITIONAL SHEAR TRANSFER ELEMENTS AS REQUIRED
FRAMING MEMBER PARALLEL TO DECK
OPEN WEB JOIST TRANSVERSE TO DECK

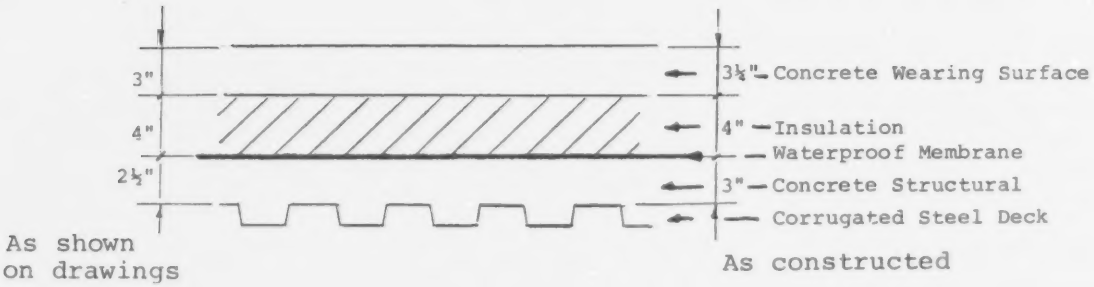


Figure 2

Thus, vertical loads are supported by the composite metal/concrete deck, and carried to the joists which, in turn, carry the loads to the beams, which transfer them to the columns and thence to the foundations.

Some of these elements perform other functions in the resistance of horizontal loads due to wind and earthquakes, but these will not be discussed here, as these loads were not present at time of collapse.

A significant feature of the Gerber system, which will be referred to later, is that the beams pass over the tops of the columns.

If the beams were simply to span between columns, they could fit between them, resting on seats attached to the sides of the columns, which then extend to the tops of the beams.

3.0 POSSIBLE FAILURE MODES OF THIS STRUCTURE

Each of the elements discussed above could fail in various ways.

However, the photographs and the debris at the site appear to indicate that failure occurred in one of the main beams such as CDEFG in Figure 1, and attention will be limited to that member.

Furthermore, it appears to have failed over a column, at a location such as D in Figure 1 (page 71).

There are several ways in which failure could have occurred:

3.1 Bending

Suppose one breaks a piece of wood across one's knee. The top fibres of the wood will be stretched and the bottom ones will be compressed. That is the condition of the beam at D.

The top flange is in tension, and may fail by tearing across the beam. The bottom flange is in compression, and may fail in *crushing*.

However, it will be recalled that elements in compression can fail by instability or *buckling*.

Thus, if the bottom flange of the beam is free to spring out sideways, it may do so while the stresses are still quite low, and thus evade its responsibility to carry the load.

Now, somewhere between D and E, the bottom flange goes from tension to compression, and the top of the column may or may not provide the constraint needed to prevent the bottom flange from jumping out to the side.

As a result, it is difficult to estimate the load which will cause instability failure. It depends upon the "unsupported length" of flange which is in compression, as well as on the compressive stresses.

These modes of failure can be avoided by making the beam big enough, and/or by providing lateral restraint for the compression flange by some means. (Note that the top flange is in compression at E, but it is restrained from sideways movement by the trusses that are welded to it.)

3.2 Shear

In contrast to the forces along the line of the member associated with bending, the beam can fail by the action of the transverse forces.

Imagine breaking a bar over one's knee. It may shear off either side, leaving a short piece on your knee, and two separate pieces in your hands.

In the steel beam, this would manifest itself by a diagonal tear in the web emanating from the bottom flange at the edge of the column, proceeding upwards from D in the general direction of C, at about 45 degrees.

This type of failure is generally avoided in the manufacturer's design of the beam, by making the web thick enough to prevent it. But, shear is normally checked by the designer.

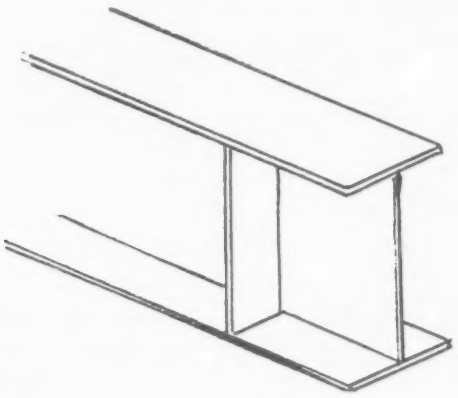
3.3 Buckling or Crippling of the Web

Immediately over the column, the web is carrying compressive stresses in the vertical direction, as the column reaction is introduced into and distributed through the beam.

Again, this immediately raises the spectre of instability: the web may buckle into one or more horizontal folds, concertina fashion, to allow the bottom flange to be pushed up into the beam.

This can be avoided by adding a vertical stiffener: a piece of steel sticking out at right angles to the web, reaching from the bottom to the top flange, and welded into place.

It will provide lateral restraint to the web, preventing it from springing out sideways.



3.4 Instability of the Beam-Column System

Recall the feature of the Gerber system alluded to above: the beam sits on top of the column.

Thus the bottom flange of the beam and the top of the column may move out to the side together, in a buckling mode.

The column is then bent over to the side and the beam is twisted as the bottom flange moves out from under the top flange, and instability failure occurs although stresses may yet be quite low.

This type of failure can be avoided by restraining the top of the column from moving sideways.

If there is a joist approaching the column from the side on the column line, a brace from the bottom chord of the joist to the end of the column will perform this function.

4.0 DESIGN OF THIS TYPE OF STRUCTURE

It is impossible to calculate exactly the capacity of a structure with respect to each of these failure modes, because, there is, in fact, no one value.

There are a number of influences, some of which vary with time, which affect the result.

These include temperature variations, slight inaccuracies in manufacture, erection tolerances, variations in material properties, stresses locked into the materials by the manufacturing processes, and so forth.

What one can say is the most probable capacity is such and such, and the range of variability about this value is likely to be such and such.
Similarly, it is impossible to say what maximum load will apply to a structure in its lifetime.

Of a thousand exactly similar structures which stand for 50 years, no two will experience exactly the same loading history, so it is impossible to predict what will happen in any one case.

Once again, we can simply say that, for a parking area, for example, the most probable maximum load will be such and such, and the variability about this value is likely to be such and such.

In view of these uncertainties, the design process is intended to provide a most probable load capacity which exceeds the most probable maximum load by sufficient margin to ensure an acceptably low probability of failure.

Absolute certainty that a structure will never fail cannot be guaranteed, except at prohibitive cost.

5.0 ANALYSIS OF FAILURES — GENERALLY

In determining the mode in which the structure actually failed, the following process is followed.

The actual load which was acting at the time of the collapse is determined as accurately as possible.

The capacity of the members with respect to each of the failure modes is calculated as accurately as possible.

Theoretically or ideally, the capacity for all modes except one will exceed the applied load. For that one mode, the capacity will equal the applied load. This mode is the failure mode.

In practice, because of uncertainties, the answer is not likely to be so clear cut. One or more modes may have calculated capacities less than or only slightly above the applied load.

Judgment may be required to assess the probable mode of failure.

6.0 LOADS

Loads are classified as "Live Loads," which are movable loads which may or may not be present, and "Dead Loads," which are the fixed loads arising from the weights of the structural and architectural components or elements themselves.

6.1 Live Load — Service Load

The construction drawings call for a live load of 50 pounds per square foot (psf). This is in accordance with the parking deck design loads of the National Building Code.

The code allows this to be reduced progressively if the tributary area for a member exceeds 860 square feet, because the probability that a larger area will be overloaded in its entirety is reduced.

The area tributary for consideration of bending in the subject beam at column line W110.84 is 900 square feet. For this area the reduction is minimal.

The design load is seldom reached in actual use.

Based on information from the Insurance Corporation of British Columbia, the passenger cars weighed an average of 3,100 pounds.

The parking stall is 8.5 feet × 18 feet or 153 square feet. The load in the stall is, therefore, about 20 psf.

If the stalls are full, the aisles are usually not, so the average load is normally less than 20 psf.

However, if we consider that fully-loaded pickup trucks, tow trucks, snow-clearing equipment, piled-up snow, etc., are possibilities, then the 50-psf-code requirement is reasonable.

6.2 Live Load — at Collapse

At time of collapse, evidence is that all car spaces in the tributary area were filled. While it is possible that one or more cars or people were moving through the aisles, we have no such evidence.

We have assumed that the average weight of 3,100 pounds was distributed 60 per cent on the front wheels and 40 per cent on the rear wheels with an average wheel base of eight feet six inches.

It should be noted that the live load at time of collapse was lower than that required by the code.

6.3 Dead Load

The design engineer's structural drawing S5.3 calls for a dead load of 80 psf superimposed on the open web steel joists plus an allowance for mechanical loads and loads suspended from the roof. Not called for, but implied, is an allowance for the weight of the beams and joists themselves.

Suspended loads include lighting, sprinklers, ventilation ducts, fans, and various walls, bulkheads, canopies, signs, etc. required by the supermarket tenant.

In the area tributary to the collapse, there were no walls and we have assumed five psf as a maximum value at the time of collapse.

Because the tenant could rearrange the store as marketing practices change, an allowance of up to 10 psf might have been prudent in the design.

The weight of the joists and beams amounts to six psf.

Figure 2 (see page 72) shows a cross section of the deck. The dimensions on the left are those that are given on the construction drawings. Those on the right are the average of measurements taken after collapse.

The extra one quarter inch in the upper slab and one half inch in the lower slab weigh nine psf. The actual dead load is:

Topping	39
Insulation	1
Membrane	3
Structural Slabs	46
Joists and Beams	6
Hung Loads	5
	100 psf

This value may vary up or down by five psf but is used by us in calculations for forces and moments at collapse.

The foregoing outlines the concepts and data required.

Below we examine the loads and capacity of the beam at the location and at the time of collapse.

7.0 BENDING

7.1 General

Photographs taken during the collapse clearly show a failure in the beam or in the beam-column assembly.

In this section, we analyse the possible bending failure of the beam in the area over the column.

Here the bending of the beam before failure was concave downward. This is often called negative bending. The top flange was in tension and the bottom in compression.

A moment is the product of a force times a distance. For example, if a 10-pound weight is hung from the end of a five-foot-long cantilever the moment M = 10 lb. × 5 feet = 50 ft. lb. at the support.

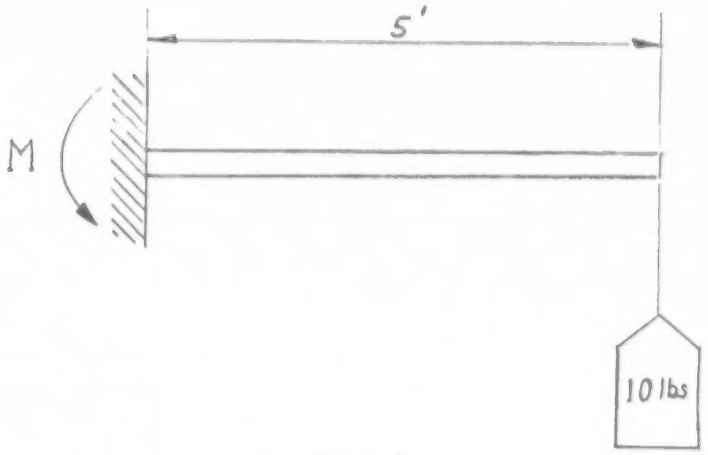


Figure 3

Similarly, in the subject beam, the loads existing at the time of collapse create bending moments M at each section along the beam.

Depending on the size and shape and lateral support, a given steel beam has a given capacity or moment resistance M_r.

If the applied moment M is less than the moment of resistance M_r, the beam does not fail in bending.

7.2 Moment Capacity M_p and M_r

The beam involved is a 24 inches deep "wide flange" beam weighing 76 pounds per lineal foot or a W24 × 76.

The yield strength of a sample taken from the flange of the actual beam after failure was 48,900 psi. For this beam, the theoretical value of resisting moment at collapse is the "plastic moment" M_p 815,000 foot pounds or 815 feet kips (one kip = 1,000 pounds).

A performance factor (ϕ) is applied to the theoretical strength to take into account the variability of the material properties, dimensions and workmanship, and other uncertainties in the prediction of strength. For beams, ϕ is normally taken as 0.9, i.e. the moment resistance $M_r = \phi M_p = 0.9\ (815) = 734$ ft. kips.

As an example of the uncertainties, page 133 of the Canadian Institute of Steel Construction publication, Limit States Design in Structural Steel, states ". . . recent studies indicate that, theoretically, ϕ should be taken as 0.84 for members failing by elastic lateral buckling."

It goes on to state that for a W24 × 76, values from 0.88 to 0.83 may apply.

We can say, therefore, that a laterally supported W24 × 76 with a yield strength of 48,900 psi has a theoretical failure moment $M_p = 815$ K ft. but it may fail at $M_r = 734$ K ft.

7.3 Reduced M_r — Unsupported Compression Flange

For all stresses up to failure, a structural element is stationary.

For this static condition, all forces or moments acting on a small section of a beam must balance. Externally applied moments are balanced by internally developed moments.

The internal resisting moment M_r is the result of a Tension force T in the top flange of the beam and an equal Compression force C in the bottom.

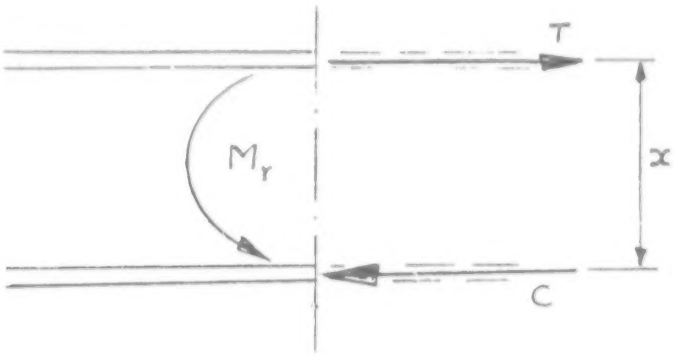


Figure 4

C and T act with a moment arm X to create M_r as shown in Figure 4.

A short or laterally supported beam can develop equal values of C and T. If loaded to failure, it will fail either by stretching of the top flange or crushing of the bottom flange.

In a beam that is long and not laterally supported, the tension capacity T is not decreased, but, because of the tendancy to buckle, the compression capacity C is.

When the compression capacity is reduced, the moment resistance capability is proportionately reduced.

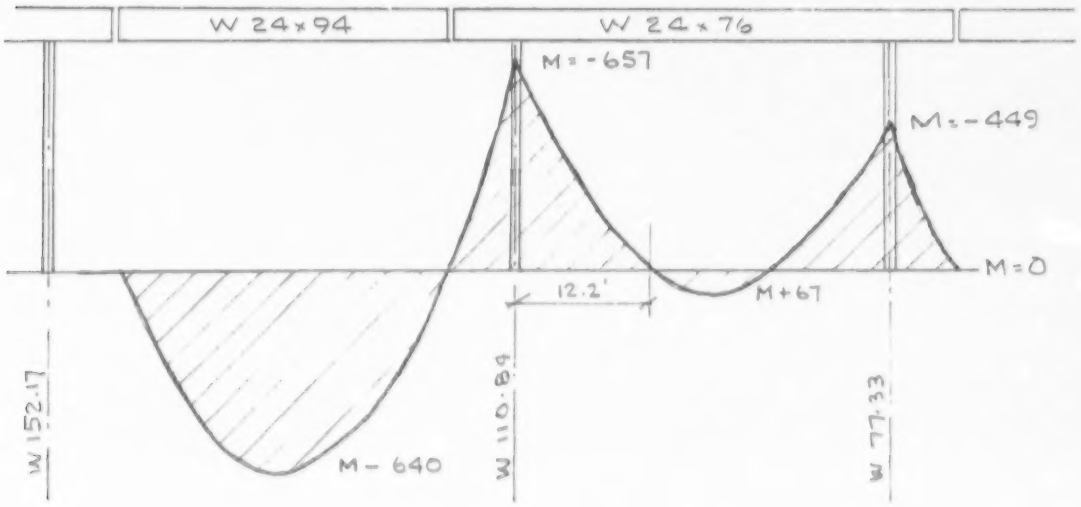


Figure 5

In the subject beam shown in Figure 5, failure occurred at line W110.84.

The top flange is welded to the joists which, in turn, are welded to the steel deck. It has, in effect, continuous lateral support.

The bottom flange has firm lateral support at the easterly column (line W77.33) where the bottom chords of the joists are extended horizontally to the column, preventing lateral movement of the column and of the bottom of the beam.

Near the column further to the west (line W152.17), a degree of support is given by angled extensions of the joist lower chords fastened to the lower flange of the beam.

Some support is also given by the column (line 110.84). This is partial support only, as the column top can (and did) move laterally to the south.

These lateral supports, are all external to the beam. Study and experience has shown that an internal support can be assumed from the portions of the beam length where the lower flange is in tension.

In the simple column discussed in 2.2 (a) above, there is an easily determined unsupported length L_u. In this beam, the determination of L_u is not so clear.

At the time of collapse, the bending moments in the cantilever beam were as illustrated in Figure 5b with negative moments at the two supporting columns and positive moment over the centre of the span starting slightly over 12 feet from the column.

If the beam had positive lateral support at the column, the unsupported length would be 12 feet. This is the lower limit for L_u.

Because the beam lacks positive restraints at the column, the effective unsupported length is greater.

We could assume that the positive moment in the adjacent supported W24 × 94 beam provides support and that the column does not.

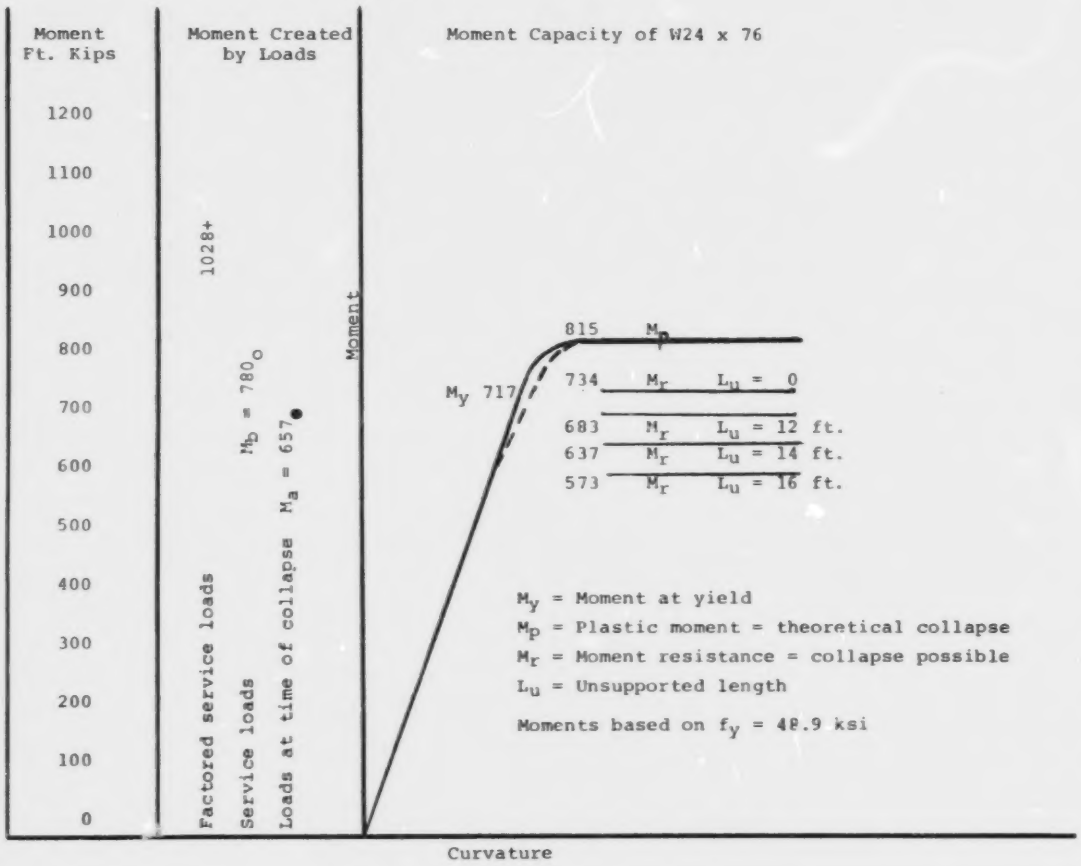
For this upper limit assumption, L_u becomes 18 feet. The effective L_u lies somewhere between 12 feet and 18 feet.

Figure 6 illustrates the values of M_r for differing values of L_u.

7.4 Comparison of Moment Due to Loads and M_r at Collapse

From the loads in sections 3.2 and 3.3, the calculated bending moment at the centre line of the column at W110.84 was 657 ft. kips.

From Figure 6, it can be seen that a bending failure could occur if the effective L_u reached about 13 feet under loads existing at time of collapse.



W24 x 76
MOMENTS CREATED BY LOADS
COMPARED TO
MOMENT RESISTANCE CAPACITY

Figure 6

7.5 Comparison of Moments Due to Service Loads and M_r

Code parking loads would have resulted in a higher moment than existed at collapse.

For use as a parking deck, the service live loads in section 3.1 together with dead loads in section 3.3 must be considered.

In the comparison in 7.4, we compared the maximum capacity moment and the actual applied moment. We did not consider the requirement of a "factor of safety" in loads.

The applied loads must be "factored up" or increased by a load factor and the total factored moment M_f created by loads must be less than M_r, the moment of resistance.

The load factors required are:

— For live load factor = 1.5, i.e. the actual design load must be increased 50 per cent. This requirement recognizes that an unusual condition may occur.

For example, someone may assemble personnel for a group photo. People crowded together weigh considerably more than the 50 psf live load specified.

— Dead load factor = 1.25. The actual dead load is increased 25 per cent, a smaller increase because it is believed that dead load can be more closely predicted.

Increased loads can result, for example, from extra concrete deck placed at mid span to compensate for deflection of supporting beams, or of supporting steel deck.

Applying load factors to the service loads results in a factored moment $M_f = 1028$ ft. kips, well above M_r for the W24 × 76. A suitable beam, if the bottom flange is laterally supported, would be a W30 × 99.

7.6 Other Considerations

Analysis is unable to deal precisely with such considerations as:

1. The maximum negative moment assumed at the centre line of the support probably does not occur because the support has, in fact, a width.

   The column is 12 inches wide and this effect is increased by the cap plate. The effect is to change the shape of the moment curve shown by a solid line in Figure 7 to that shown by a broken line.

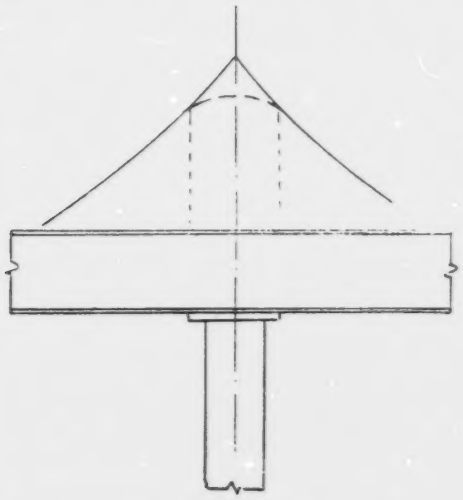


Figure 7

2. The beam was fastened to the cap plate by four bolts.

   One normally assumes that the bolts fill the holes as the material yields in compression, but the presence of bolt holes causes local increase in stress.

3. The load on the beam comes from adjacent spans of 46.5 and 40.33 feet. The result is a tendency for the beam to rotate which increase stresses.

   This tendency is partially resisted by applying moments to the joists through the welded connections.

4. Because residual strains are created in wide flange steel beams when they are fabricated, the tips of the compression flange may start to yield at stresses about 0.7 M_y.

   This will allow movements to occur which effect buckling and the precise level of collapse.

In normal design procedures, factors such as these are accounted for in the safety factors, but they are impossible to quantify with any precision in calculating the exact collapse load.

7.7 Bending Summary

1. The W24 × 76 is too small for bending even if it had been fully supported against lateral buckling. A suitable section, which still requires lateral support, is a W30 × 99.
2. Failure may or may not be in bending alone.

8.0 SHEAR

As noted earlier, shear failure of manufactured steel beams is unusual, since the manufacturer designs the web thickness so that shear failure will not occur under normal conditions.

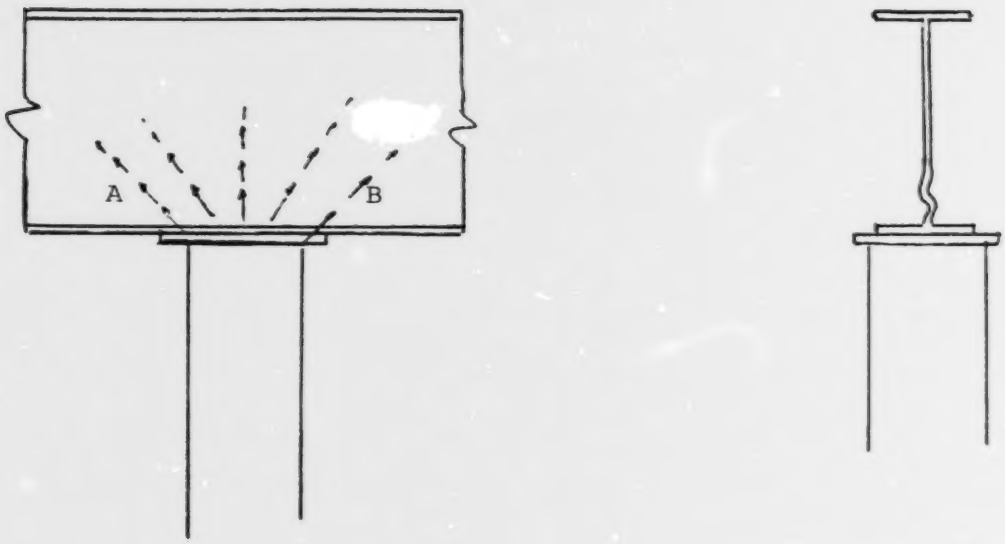
In the present case, the calculated shear force at the time of collapse was 124 kips, while the shear capacity of the beam was 302 kips, reduced to 272 kips by the ϕ factor to account for uncertainties.

Thus, shear was clearly not a factor in the collapse. Indeed, the photographs confirm this conclusion.

9.0 WEB CRIPPLING

9.1 Concept

The column provides a vertical reaction to support the beam. This vertical force spreads up into the beam, so that the web of the beam is subject to vertical compressive forces which are high at the bottom but which decrease towards the top flange.



Recalling the general problem of stability under compressive forces, the possibility exists that the web can buckle, forming waves parallel to the beam flange.

9.2 Analysis

The column reaction is found to be 200 kips. The force required to cause crippling of the web as shown in the figure, depends upon the length AB over which the force is distributed when it gets up into the web. Thus, here again, some judgment is required.

Assuming that the load spreads upwards at 45 degrees from the outside face of the column (a common assumption), one can calculate a capacity of 396 kips, based on the specified yield strength of 44 kips. The value reduced by the ϕ factor, to account for uncertainties, is 356 kips.

If we assume, very conservatively, that the web could cripple over each column wall (at A and B) independently, with a portion over the centre of the column which remained unaffected, we would get a value of 236 kips, reduced by the ϕ factor to 212 kips.

9.3 Conclusion

Even by the most conservative analysis, crippling of the web was apparently not the trigger causing this collapse.

10.0 INSTABILITY OF THE BEAM-COLUMN SYSTEM

10.1 Concept

In the discussion of bending above, the importance of lateral support for the compression flange was emphasized.

In determining the bending capacity, various lengths of the flange were considered to be unsupported.

It is possible that the beam-column system may become unstable: the column may move out sideways to escape the vertical compression forces, at the same time allowing the bottom flange of the beam to move out sideways to escape the horizontal compression forces.

This mechanism would also involve twisting of the beam as the bottom flange moves out, with the top flange held in line by the joists and the roof deck.

It would also involve local distortion of the beam as it is forced to conform with the orientation of the column head, although it is generally twisted the opposite way.

10.2 Analysis

The results of three attempts to analyse this condition are available to the Commission: one made by ourselves, one made for us by Jones Kwong Kishi, and one made by the Workers Compensation Board.

The analyses are rather complex and are strongly influenced by the assumptions that are made, as well as by the factors listed in section 7.6 under bending.

Thus, they must be regarded as indications rather than as definitive statements of what occurred.

Note that, in doing an original design, one does not attempt such an analysis. If the possibility of this condition is thought to exist, it is prevented by the addition of braces.

10.3 Conclusion

These analyses show that this instability mode, which is clearly shown by the photographs to have occurred by that stage, could have been the primary mechanism for the collapse, but that is not likely.

On the other hand, the analysis made by Jones Kwong Kishi suggests that rotational restraint provided to the top flange of the beam by the joist connections could have been sufficient to prevent buckling.

However, it is quite likely that the high bending stresses allowed the instability mode to occur more easily, i.e. that the actual mechanism was a combination of bending and instability.

11.0 CONCLUSIONS

Failure involved at least two modes.

Mode A

A beam bending failure with buckling of the lower flange. The effective unsupported length of the lower flange is not less than 12 feet and may be 18 feet or more. At 13 feet, the moment capacity is reduced and equals the moment created by the April 23 loads. This is the most probable prime cause of failure.

Mode B

Buckling of the beam column assembly. Our calculations indicate that this is a less probable prime cause but, together with Mode A, may have resulted in simultaneous failure in both modes.

The beam was greatly underdesigned.

The beam column assembly lacked essential lateral supports.

Failure was unavoidable. If not under April 23 loads, then under full service loads.

APPENDIX A

Bending Calculations

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COMMISSIONER INQUIRY
STATION SQUARE DEVELOPMENT
BURNABY

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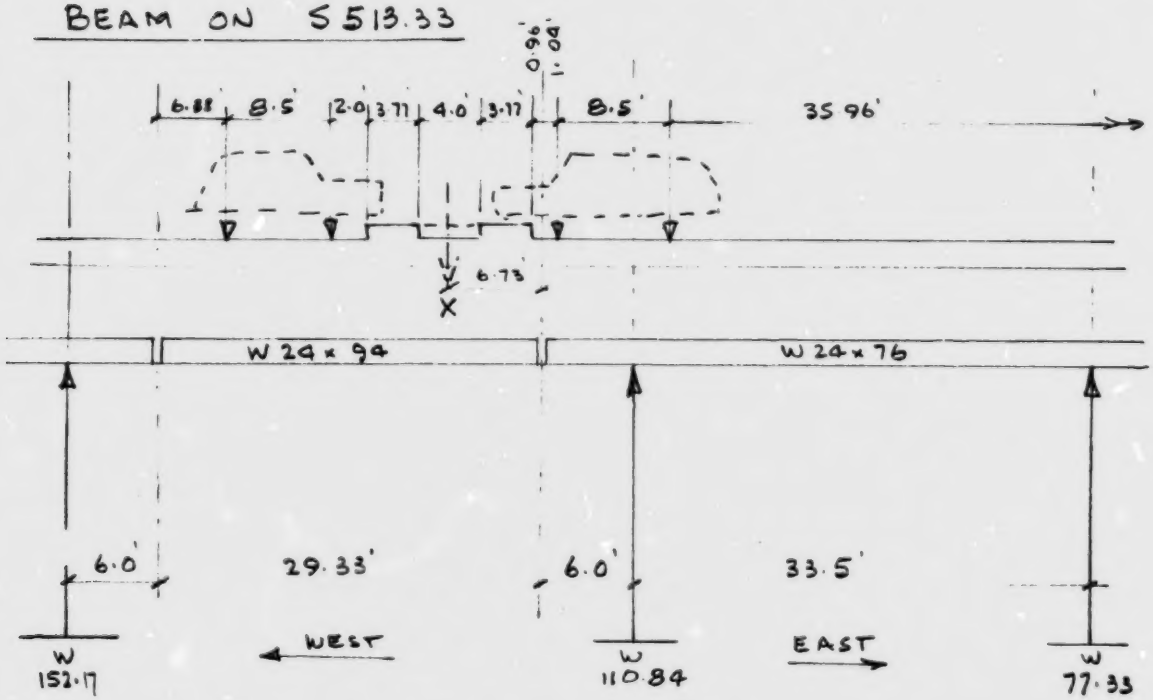
Page 1.

BEAM ON S513.33



LOADING AT TIME OF COLLAPSE

Tributary width of deck 46.5 + 40.33/2 = 43.42'

		Working Loads	Factored loads
Dead Load : 100 psf			
Deck	43.42 x 100	4342 plf	
Extra at sidewalk	consider as point load at x		
	2x43.42 x 3.77x 75	24.56 K	

Live Load 3100 lb car 60% front axle 8'-6 stalls

Front	.6(3100) x 43.42 - 8.5	9.50 K
Rear	.4	6.33 K

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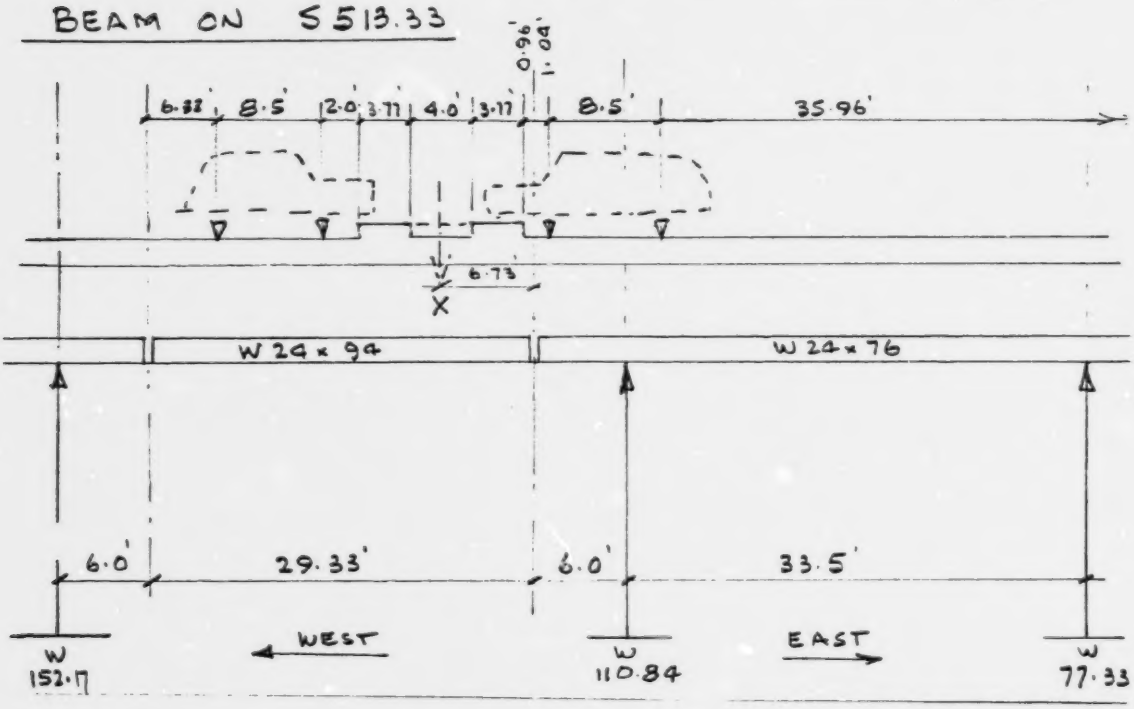
Page 1A

BEAM ON S 513.33



DESIGN LOAD

Tributary width 43.42'

area 43.42 (29.33) = 1273 sq ft

Assume ① Dead Load 100

Specified 80 superimposed

structure 6

hung 10

96

① sidewalk as revised. ie 24.55 ℄

② Live Load 50 reduced per 4.1.6.3(5) = 49 psf

Total 145 psf

Note. It is reasonable to ignore 100 psf requirement for walkway

Tributary area $\left(\frac{29.33}{2}+6\right)43.42 = 900$ sq ft

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Page 2.

SUSPENDED BEAM — WORKING STRESS

Dead	R_W =	4.34 × 29.33 ÷ 2 =	63.65	R_E	= 63.65
		24.35 × 6.73 ÷ 29.33 =	5.63		24.55×22.60÷29.33 = 18.92
		Σ D.L	69.28		82.57
Live		9.5 × 14.5 ÷ 29.33	4.70		9.5×14.83÷29.33 = 4.80
		6.33 × 23 ÷ 29.33	4.96		6.33 × 6.33 ÷ 29.33 = 1.37
			9.66		6.17
Live + Dead	R_W =		78.94	R_E =	88.74

Point of zero shear $\frac{78.94 - 6.33}{4.34} = 16.7 > 14.83$

Max M at front wheel $M = 78.94(14.83) - 6.33(8.5) - \frac{4.34(14.83)^2}{2}$

Required M = 640.7 Kip feet

Capacity of W 29 × 94 M_R = 537

∴ overstress $\frac{640.7}{537} = 1.193$

19% overstressed

SUSPENDED BEAM — LIMIT STATES

R_W	dead	69.28 × 1.25 =	86.6	R_E	82.57 × 1.25 = 103.2
	live	9.66 × 1.5 =	14.5		6.17 × 1.5 = 9.3
			101.1 Kips		112.5 Kips

Point of zero shear $\frac{101.1 - 14.5}{4.34 \times 1.25} = 16$

Max M $M_B = 101.1(14.83) - 6.33(1.5)(8.5) - \frac{4.34(14.83)^2}{2}1.25$

Required = 822 Kip ft

Capacity 838 kip ft. OK.

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FOR FULL DESIGN LOAD

SUSPENDED BEAM. WORKING STRESS

		R_w	R_L
Dead	(from 1)	69.28	82.57
Live	$\frac{43.42 \times 29.33 \times 45}{2}$	28.65	28.65
Total.		97.93 K	111.22

Point of zero shear $97.93 \div (6.30 \times 43.42) = 15.55$

$M = (97.93 \times 15.55) - \left(\frac{6.30 \times 15.55^2}{2}\right)$

$= 761$

	Required	provided
M_e	537	

SUSPENDED BEAM LIMIT STATES

		R_w	
dead		86.6	103.2
live	28.65(1.5)	43	43
		129.6	146.2

Point of zero shear $129.6 \div (6.30(1.5)) = 13.72$

$M = 129.6(13.72) - \frac{6.30(1.5)(13.72)^2}{2}$

$= 859$

$M_e = 838$

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Page 3.

CANTILEVRED BEAM. WORKING STRESS.

Moment at W 110·84 at time of collapse.

from suspended beam	88·79 × 6	=	532·4
front axle	9·5 × 4.96	=	47·1
dead load	$\frac{4.34 \times 6^2}{2}$	=	78·1
			657 6 Kip ft

Moment capacity W24×76 if supported. 426 Kip ft

426 is based on stress = 0.66 yield stress

∴ 657 = 0.66 × 657 / 426 = 1.01 yield stress

i.e. gross overload at, or approaching, yield even if full lateral support.

[Moment at W 110·84 if design loads applied.

from susp beam	111·22 × 6	667.6
from cantilever	$.145(43.41) \times \frac{6^2}{2}$	113·3
		780·9 Kip ft]

LIMIT STATES

Moment at time of collapse

from suspended beam	112·5 × 6	=	675.0
front axle	9·5 × 1·5 × 4.96		70·7
cantilever	$4.34 \times 1.25 \times \frac{6^2}{2}$		97·6
			843·3

Moment capacity M_R = 660.0 Kip ft

[Moment at W110 84 if design loads applied

from suspended beam	146·2 × 6	877
cantilever	$.194(43.42) \times \frac{6^2}{2}$	151
		1028 Kft]

96×1.25 }
49×1.5 }

All the above assumes lateral support and shows

At failure M was 657 kft

Capacity safe was 660 kft
without ϕ factor 733

If loaded to design load M would be 780.9 and failure would occur even with lateral support

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Page 4.

Effect of Planter on beam east of 77.33

Planter
22.0 | 18.33

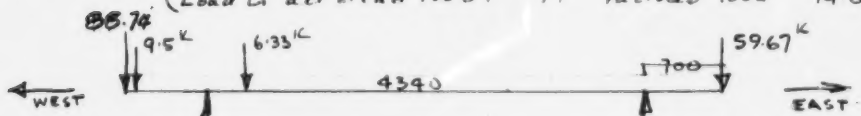
Assume 2'-0 x 5" Earth 6S wide @ 200 psf 1300
Walls 2 x 2 x 60 240
1540 lb / ft.

Load to beam $\frac{18.33}{40.33} \times 1540 = 700$ lb/ft

This extra load is applied to the hull suspended beam and to the hull length of the cantilever.

Reaction to the east end of the cantilevered beam (length from slip drwg 575A)
from suspended slab $\frac{23.67}{2} \times (700 + 4342) : 59.67^{K}$

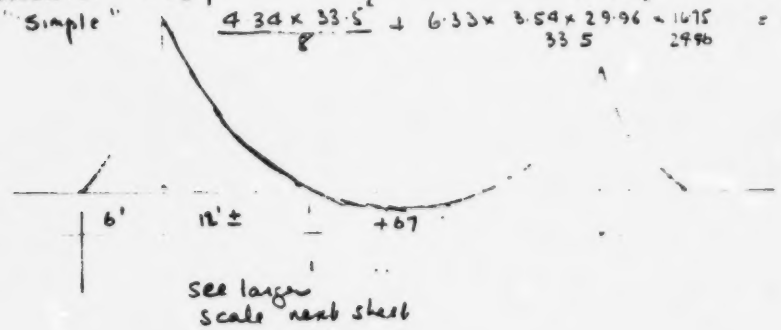
(Load is all dead load. ∴ factored load 74.6^{K})



Rw	88.74 x	39.5	= 3505	RE	− 88.74 x 6	= 532 −
	9.5 x	38.96	= 365		− 9.5 x 4.96	= 47 −
	6.33	29.96	= 190		6.33 x 3.54	= 22
	4.34 x	41.5 x 16.75	= 3016			3016
−	.7 x 6 x 3		= 12 −		.7 x 6 x 36.5	= 153
−	59.67 x 6		= 358 −		59.67 x 39.5	= 2357
			6706			4989
		÷ 33.5	= 200K			149

Moments "Simple" −657 −449

$\frac{4.34 \times 33.5^2}{8} + 6.33 \times \frac{3.54}{33.5} \times 29.96 \times 16.75 = 620$



see larger scale next sheet

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Page 5.

MOMENT REDUCTION WITH INCREASED Lu.

Beam is W 24×76 (W 610×113). Class 1 for f_y = 44 Ksi (A S.6)

A CAN3-S16·1M78 E = 200 000 Mpa (A 1-1B) = 29 000 Ksi

B Limit State Design in SS G = 77 000 " = 11 200 Ksi

d = 23.92 in t = .68 in

b = 8.99 " w = .44 in

Z_x = 200 in³ J = 2.68 in⁴

I_y = 82.5 in⁴ $C_w = \frac{I_y (d-t)^2}{4}$ = 11100 in⁶ (A 8.10)

S_x = 176 in³

For f_y = 48.9 Ksi $M_p = 200\left(\frac{48.9}{12}\right)$ = 815 ft Kip

ω = 1.0 assumed (see A 1-40)

no effective lateral support at one end of Lu

For Lu = 12'

$$M_u = \frac{\pi}{\omega L}\sqrt{E I_y G J + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

$$= \frac{\pi}{1(144)}\sqrt{29000 \times 82.5 \times 11200 \times 2.68 + \left(\frac{\pi 29000}{144}\right)^2 82.5(11100)}$$

$$= 0.028\sqrt{7.18 \times 10^{10} + 36.66 \times 10^{10}}$$

= 14434 in Kips = 1203 ft Kips > ⅔ Mp.

ϕ = 0.9

$$M_r = 1.15\phi M_p\left(1 - \frac{0.28 M_p}{M_u}\right)$$

$$= 1.15(.9)(815)\left[1 - \frac{0.28\ 815}{1205}\right] = 683 \text{ ft K}$$

For Lu = 14'

M_u = .0187 $\sqrt{7.18 \times 10^{10} + 26.93 \times 10^{10}}$ = 10921 in K = 910 ft K

M_r = 1.15 ϕ 815 $\left[1 - \frac{0.28\ 815}{910}\right]$ = 637 ft K

For Lu = 16'

M_u = .0169 $\sqrt{7.18 \times 10^{10} + 20.62 \times 10^{10}}$ = 8647 in K = 720 ft Kips

M_r = 1.15 ϕ 815 $\sqrt{1 - \frac{0.28\ 815}{720}}$ = 573 ft K

APPENDIX B

Biographies

C. Peter Jones, P.Eng.

BA; BASc (Civil, UBC) 1948. Registered in B.C. 1952.

1940-45: RCNVR Service Overseas & North Atlantic.

1948-52: Swan Rhodes & Wooster, Consulting Engineers, Senior Staff Engineer.

1952-74: Read Jones Christoffersen Ltd., Consulting Engineers, Director & Vice-President.

1979-86: Jones Kwong Kishi, Consulting Engineers.

1986 to date: Consultant to Jones Kwong Kishi.

Architects' and Engineers' Joint Board of Ethics 1960s; Association of Professional Engineers of B.C. Nominee on Provincial Construction Safety Enquiry Committee 1960s; Association of Professional Engineers of B.C. Nominee on Provincial Construction Safety Enquiry Committee 1982; Working Committee on Disciplinary Procedure 1985-86; Investigation Committee 1985-Present.

Vice President, Association of Professional Engineers of B.C. 1988.

Noel D. Nathan, P.Eng., is a professor of civil engineering at the University of British Columbia.

He graduated from the University of Witwatersrand in South Africa with a BSc in civil engineering in 1949, subsequently gaining an MS at the Massachusetts Institute of Technology in 1954 and a PhD from the University of Washington in 1969.

He worked for a contractor and for consulting engineers in South Africa, the US, and Canada before joining UBC in 1958.

Since then, he has been involved from time to time in investigations of engineering failures and in other consulting work.

APPENDIX IV — *Station Square Developments Inc. Vancouver, B.C.*

REPORT 28726 **JUNE 1988**

SAVE-ON-FOODS STORE STATION SQUARE ROOF COLLAPSE — FINAL REPORT

Prepared by

John Rapson, P.Eng.
of
Sandwell Swan Wooster

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1. INTRODUCTION

The following report has been prepared for Station Square Developments Inc., a wholly owned subsidiary of Wesbild Enterprises Ltd., and the owner/developer of the Station Square complex, of which the Save-on-Foods building forms a part.

Sandwell Swan Wooster are consulting engineers covering a broad range of disciplines including structural engineering. The senior engineer primarily responsible for this investigation has extensive experience in similar assignments as outlined in Appendix F.

The report addresses the probable causes of the collapse of part of the rooftop parking deck of the Save-on-Foods building. The collapse occurred at approximately 9:15 a.m. on Saturday, 23 April 1988, on the preview opening day of the store.

A section of roof some 6,400 square feet in area failed and twenty vehicles parked above fell into the opening.

As has been previously documented, some prior warning of the failure was given by the rupture of a sprinkler line, and enough time elapsed to evacuate the area.

2. DESCRIPTION

The subject building is a single storey supermarket structure forming part of the Station Square development, which in turn forms a component of Burnaby's Metrotown complex. The development is located directly south-east of the intersection of Kingsway and Mackay Rd. The Save-on-Foods building is approximately retangular in shape, some 400 feet in a north-south direction and 255 feet in an east-west direction. To the west lies a multilevel parking structure, part of the complex, and the supermarket store connects to this via two vehicular ramps which give access to the rooftop parking, and two pedestrian walkways, which connect to a mezzanine. For further details to the key plan, Drawing No. B-28724-SK1.

The building is single storey, other than for the small mezzanine along the east wall, and approximately 25 feet high from main floor to the underside of roof deck. The roof top is utilized for parking, containing spaces for some 235 vehicles. Vehicles such as large vans, trucks, R.V.'s etc., whichs could overload the deck, are prevented access to this parking deck by height controls in the adjacent parkade, from which access is gained via the ramps. Weight limit signs are also posted.

3. CONSTRUCTION

The building is of generally conventional construction and materials.

3.1 Foundations and Walls

All foundations and some exterior walls are of cast-in-place concrete using conventional spread foundation design. Foundation depth is generally quite shallow with occasional deeper zones as required by site conditions or the existence of previous buried construction or backfilled areas. In the collapse area the foundations were shallow, bearing on dense glacial till subsoil at a depth of about 4 feet below original grade. This till has a documented safe bearing capacity of some 10,000 lbs. per square foot.

Some exterior walls are of concrete block masonry, and the west or front face of the building is glazed full length.

3.2 Rooftop Parking Deck

This is a sandwich construction comprising a nominal 3 in. thick fibre welded wire fabric reinforced concrete wearing course overlying a 4 in. thick "Roofmate" styrofoam insulation, which is laid on a nominal 2½ in. concrete over 1½ in. composite metal deck which comprises the structural slab. This slab also contains welded wire fabric reinforcement and is waterproofed with a liquid membrane covered by two layers of protection board on its upper surface, beneath the insulation. A pedestrian walkway, some 11 ft. 6 in. wide, extends from north to south along the roof centreline and there are a number of concrete planters at various locations over the roof, in addition to various mechanical and air handling equipment, enclosures, stairwells, entry foyer, etc. For a general rooftop layout see Drawing No. B-28724-SK2.

3.3 Structural Framing

The framing system is based upon a rectangular grid, containing a great number of irregularities. Dimensions are varied in both directions and many columns are located off the grid, presumably as required to suit the functional operation of the store. The area of the collapse was one in which the spans were among the largest in both directions.

The structural framing system typically comprises open web steel joists, 42 in. in depth and at centres varying between 4 ft. 0 in. and 6 ft. 6 in. These run in a north-south direction and are carried on lines of beams which run in an east-west direction. The joist spans vary generally between 36 ft. 0 in. and 46 ft. 6 in.

The beam lines run between east and west walls and are carried on columns spaced generally between 33 ft. 6 in. and 40 ft. 0 in. on centres. The beam layout is based on the 'cantilever/drop-in' concept wherein alternate beams span over the tops of two columns and extend as cantilevers on each side, while the infill beams are hung from the ends of the cantilevers. For further details see Drawing No. B-28726-SK3.

Beam sizes vary with span and loading and range between W21 × 55 and W24 × 162. Beam lines are not always level but sloped in various locations to create positive drainage on the slabs above, drains being fitted through the roof deck at the low points. Discounting these minor variations the tops of the beams were some 24 ft. above the ground level floor.

Columns are fabricated of 12 in. square hollow structural tubing, their wall thickness being either 7/16 in or ½ in. Many columns are drilled for conduit or drains and locally reinforced at such locations. The beams are carried over the tops of the columns, bearing on a column cap plate but without beam web stiffening at any location. The joist lower chords are entended to the column or to the lower beam flange along two north-south lines, but three other lines of columns are unrestrained in the north-south direction.

Along the west or front face of the structure the framing system changes to accommodate the travelator access through the mezzanine to the rooftop, and to provide extensive glazed areas.

4. COLLAPSE AREA DESCRIPTION

The collapse occurred between Baylines S466.83 to the north, S553.66 to the south, W77.33 to the east and W151.92 to the west, thus measuring some 87 ft. in a north-south direction and 75 ft. in an east-west direction, an area of some 6,475 square feet. The structural layout of this area is shown in Drawings No. B-28726-SK3 and SK4, and comprised 42 in. deep open web steel joists at 6 ft. 1½ in. centres spanning 46 ft. 6 in. over the northern section of the area, and 42 in. deep open web steel joists at the same centres spanning 40 ft. 4 in. over the southern portion. The beam line just to the south of centre comprised a W24 × 76 rolled steel beam of the cantilever type to the east and a W24 × 94 drop-in beam to the west. Beam

lines at the north and south limits of the collapse area are similar in design but vary in weight due to span differences.

The pedestrian walkway referred to in Section 3.2 crosses this area just to the west of centre and its load transfers to the east end of the W24 × 94 drop-in beam. A planter is located east of the collapse area adjacent to the southern portion; its load is partially transferred to the beams east of the collapse area.

The particular layout of the parking stalls means that vehicular loads are concentrated on the drop-in beams and the cantilever ends of the main beams, while the centre span of the main beams in this area, as the travel aisle, is lightly and intermittently loaded.

5. MATERIALS AND PROPERTIES

5.1 Topping Concrete

The 3 in. thick topping or wearing surface concrete is normal weight concrete containing chopped glass fibre reinforcement to aid in crack control and further reinforced with light welded wire fabric 6 × 6 × 10/10 for control of shrinkage cracking. Specified compressive strength of this topping is 20 MPa or 2,900 lb. per square inch at 28 days but a stronger mix was used and test strengths are in excess of 30 MPa or 4,350 lb. per square inch. Measured thickness of this slab is variable, up to a maximum of 3.9 inches and tending to average 3.25 inches. This has the effect of increasing structural dead weight by approximately 3%. The light welded fabric was generally trodden down during concrete placement and can be seen on the underside of the slabs, thus rendering it of little value either for crack control or for load distribution.

5.2 Insulation

This is 4 in. high density closed cell styrofoam, "Roofmate" manufactured by Dow Chemicals. Its high density gives it adequate compressive strength to absorb vehicle wheel loads, while the closed cell construction prevent moisture absorption.

5.3 Waterproof Membrane

Applied to the top of the structural slab this comprised a liquid polymerised bitumen coating which was then covered by two layers of protection sheet to prevent damage during subsequent construction.

5.4 Structural Slab

Composed of a nominal 2½ in. slab over a 1½ in., 22 g keyed composite metal deck, using normal concrete with a compressive strength of 20 MPa or 2,900 lb. per square inch at 28 days, this slab trended towards higher thickness than nominal, due in part to deflection of the metal deck and supporting structure during concrete placement, and also due to constructional tolerances. The average slab depth over the deck was of the order of 3 inches which again increased the structural dead load by some 6%. The metal deck was of steel conforming to ASTM A443 with a guaranteed yield stress of 33,000 lb. per square inch. A light welded wire fabric, 6 × 6 × 8/8 was placed in this slab to aid in crack control.

5.5 Open Web Steel Joists

These members support the deck, carrying their loads to the main beam lines. The joists were fabricated by Great West Steel Industries Ltd. using hot profile chords and flattened tube or flat bar webs. The material used for the chords is a proprietary section, rolled by Manitoba Rolling Mills Ltd. with a guaranteed yield strength of 400 MPa (58,000 lb. per square inch). A test sample indicated a yield strength of 61,800 lb. per square inch. (See Appendix C.)

5.6 Structural Steel Beams

Beams are of a wide ranging variety of sizes, obtained from a number of different sources. The required specification is Canadian Standard CSA G.40.21M - 300W which specifies chemical composition, a minimum yield stress of 300 MPa or 43,500 lb. per square inch, an ultimate tensile stress in excess of 80,000 lb. per square inch and requirements for ductility. Three samples taken from the failed beam met or exceeded all specification requirements. (see test results, Appendix C.)

5.7 Hollow Structural Steel Columns

The columns are 12 in. square hollow tubes with 7/16 in. or ½ in. wall thickness, specified as conforming to CSA G.40.21M - 350W, with a minimum yield stress of 350 MPa or 50,750 lb. per square inch. Since the column at the centre of the collapse area underwent severe displacement during the collapse without buckling failure, no material testing has been undertaken.

6. COLLAPSE ANALYSIS

An engineering analysis of the loads and stresses on the area at the time of collapse has been undertaken, using assessments of load based upon estimated weight and actual location of vehicles, calculated weight based upon measured thicknesses of construction materials and with reasonable allowances for the weight of hung lighting, ductwork, sprinkler lines, signage, etc. below the roof. Appendix E lists the vehicles that were on the roof in the area of the collapse. The following assumptions were made:

1. Vehicle weight average — 3,100 lb.
2. Vehicle wheelbase — 8 ft. 6 in.
3. Weight distribution — 60% front, 40% rear.
4. All vehicles parked nose in to pedestrian walkway (visible from aerial photographs taken after collapse).
5. Parking bays each side of walkway full, parking bay to east empty (visible from aerial photographs).
6. Travel aisles empty.
7. Planter to east containing 2 ft. 0 in. of soil at 110 lb. per cubic foot.
8. Upper concrete wearing surface average 3.25" thick weighing 39 lb. per square foot.
9. Pedestrian walkway adding additional load of 75 lb. per square foot at sides, 4.5 lb. per square foot at centre (due to styrofoam void form in centre).
10. Insulation, 4 in. thick weighing 1 lb. per square foot.
11. Waterproofing membrane and protection boards weighing 3 lb. per square foot.
12. Lower concrete structural slab averaging 3 in. thick over 1½ in. deck and weighing 46 lb. per square foot.
13. Structural steel joists, beams, bridging, etc. weighing 6 lb. per square foot.
14. All allowance for hung mechanical, electrical, ductwork and signage components of 5 lb. per square foot.

The vehicle loads are substiantially less than are envisaged by the B.C. Building Code as design criteria, but represent an accurate estimate of actual loads at the time of collapse.

A first analysis was carried out using elastic design techniques to determine actual forces and stresses in the members at the time of failure in order to compare against permissible stresses from the "old" CSA Standard for the elastic design of steel structures, CSA S16-1969. While this Code is outdated and much design today is undertaken under CSA S.16 M78/M84, which utilizes factored loads and limit state design, it was considered useful for non-technical readers to have an understanding of actual stress figures at collapse. (For further details see Appendix D.)

The layout of the walkway, planters and parking bays in the area is such as to transfer heavy loads and reactions to the cantilevered ends of the beam. These loads cause the beam to "hog" or arch upwards and generate severe negative bending moments in the beam over the columns. Negative bending moments result in compressive stresses in the bottom flange and lower portion of the web of the beam, with corresponding tensile stresses in the upper flange and web zone.

Under the loading at collapse, these stresses reached a maximum of some 45,100 lb. per square inch, above the guaranteed yield stress of 43,500 lb. per square inch and considerably above the maximum allowable stress, taken as 0.66 of yield or 28,700 lb. per square inch. In fact, since the maximum allowable stress is only allowable for beam flanges that are adequately restrained against lateral deflection, and this beam was unrestrained, the level of overstress was even more severe.

A second cause of beam distress could have been caused by the imbalanced loads resulting from the unequal spans on either side. These could have induced a torsional or twisting load in the beam, which could have been translated to a lateral force of approximately 1,550 lb. applied at the head of the column. Since the column head was not restrained it could have been free to move laterally under this force, which would have been sufficient to move it approximately 0.2 inches. Such lateral movement would have resulted in further compressive and tensile forces in the lower flange of the beam as a result of the horizontal bending of the flange. These stresses, caused by the force in the flange and the lateral displacement, commonly known as the P-Delta effect, added some 6,020 lb. per square inch compressive and tensile stress to the already overstressed bottom flange, pushing compressive stresses on the north side of the flange at the column head to some 51,120 lb. per square inch, above the test yield of the material.

The flange yield served only to accelerate displacement of the column; this shifted the web of the beam from vertical and resulted in the web yielding in bending. From this the rotation of the beam on its longitudinal axis began to break the joist welds at the top flange and total collapse was inevitable and imminent. It is believed that as the beam began to move downwards the joists behaved as tension members, almost as cables, for a short period before their connecting welds failed, and that this behaviour gave a little more time for the area below to be evacuated.

The foregoing postulates the most likely cause and sequence of the collapse, and is, to some extent, confirmed by the photographs taken during failure (see Appendix B). However, the levels of overstress were so high that other mechanisms could have been instrumental. The beam lower flange was so highly stressed and so unrestrained against lateral movement that it was in a potential-buckling failure mode, and indeed had been so since the concrete deck was placed. Furthermore, compressive stresses in the beam web from the column reaction were so high that the web was also in an incipient failure mode, and if the column were erected slightly off plumb, although within accepted erection tolerance, the resulting misalignment could have triggered an ever increasing P-Delta stress effect.

The beam, in the column head zone, was so highly overstressed and so unstable in the north-south direction that collapse at sometime was a virtual certainty, requiring only the catalyst of slight additional load as cause.

A second analysis was undertaken using the design provisions of CSA S16.1-M78, which forms part of the B.C. Building Code and of the National Building Code of Canada 1980, the documents under which the building design was undertaken and controlled, and which formed part of the by-laws of many municipalities, including the Corporation of Burnaby. This design code uses limit state design wherein actual loads on structures are magnified by a variable multiplying factor and the moments, shears and other resultant forces are compared against the ultimate capability of the particular member as outlined in design tables. This analysis gave the applied factored moment on the W24 × 76 beam as 848 kip.ft. If the assumed unsupported length of the beam compression flange is taken as the distance between the points of contraflexure, on the basis that lateral restraint is supplied by the slight positive midspan moment and by the end of the drop-in beam, its factored moment of resistance is 467 kip.ft., thus giving an overstress of 82%. (See Appendix D.)

A third analysis was undertaken by computer, using the "P-Frame" program for stability. This allows the modelling of a section of beam on the column under gradually increasing loads until instability and failure is reached. This model predicted failure at a column load of 206,700 lb. which compares closely with the calculated load at failure of 209,630 lb. (Appendix D).

7. REASONS FOR COLLAPSE

The direct causes were, as has been outlined, the combination of excessive stress levels and lateral instability. A series of occurrences during design and construction also served to aggravate load conditions; the pedestrian walkway across the centre of the roof was more than doubled in width, and its weight increased by 250% as a result of a field change, the planters on the roof were added and the increased thicknesses of concrete actually poured resulted in an estimated dead weight increase of some 10%. As has been noted earlier, the pattern of loading on the roof is such that worst case conditions are created on the cantilever beams in the collapse area.

Certain columns were provided with lateral restraint between the bottom chord of the open web joist on the column line to the column, or to the beam lower flange where the joist was not immediately on the column line. The lack of lateral restraint to other columns was apparently addressed by the structural engineers, who added various notes to a fabrication drawing, the key one reading "Provide bottom chord extensions to column or U/S beam adjacent to column where applicable on at least one side noted as BCE". The note BCE was placed on the drawing at 29 locations, one of which was at the column where failure began.

The 29 bottom chord extensions requested were not in place when the building was completed, and at the time of writing we are not aware of the reasons why the notations were not acted upon.

The omission of the extension at the crucial column location had a small, but possibly significant effect upon the course of events; had it been in place on the day of collapse the beam, although very severely overstressed, might not have been able to shift sideways and collapse as a result.

Nevertheless, the levels of overstress and the lack of further beam flange restraint in the compression zones would have triggered a failure at some time, perhaps under conditions of greater load such as during a snowfall, an earthquake, or other abnormal loading. Furthermore, the written instruction to install the chord extensions on one side only renders their capability suspect, since they could, in this manner, transmit horizontal compressive forces into the lower chord of the open web joist, which it would not be designed to withstand.

8. SUMMARY

Bending moments, shears, and stresses in the cantilever beam, in the area of the column head were substantially in excess of those permitted by governing Codes, even under the loads present at the time of failure, which were less than Code design criteria.

Lack of restraint to the column head in the north-south direction resulted in beam lower flange instability, leading to failure caused by flange buckling in the already overstressed area adjacent to the column head.

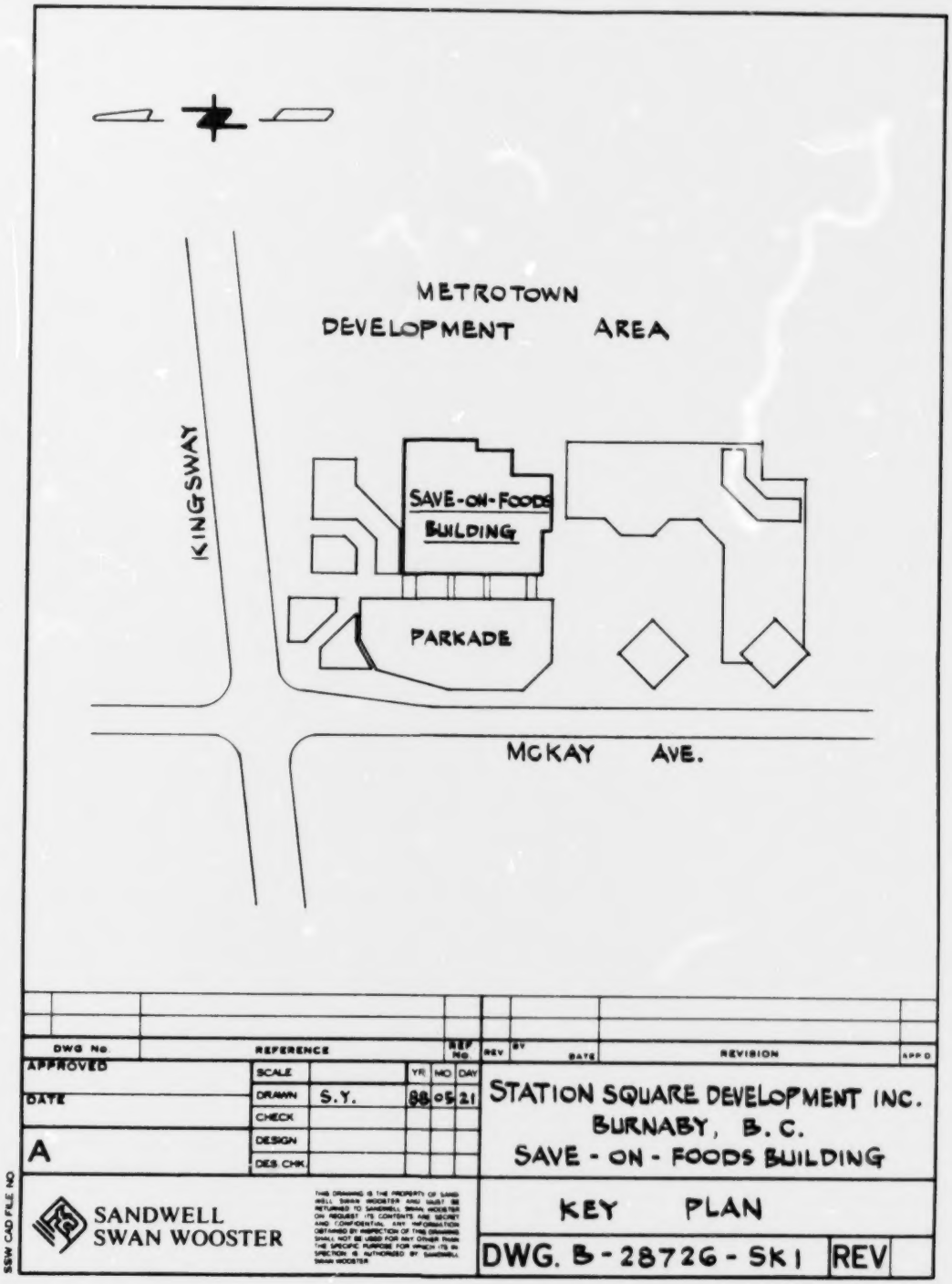
Flange buckling resulted in web buckling, almost instantaneously, as a result of high compressive stress levels within the web.

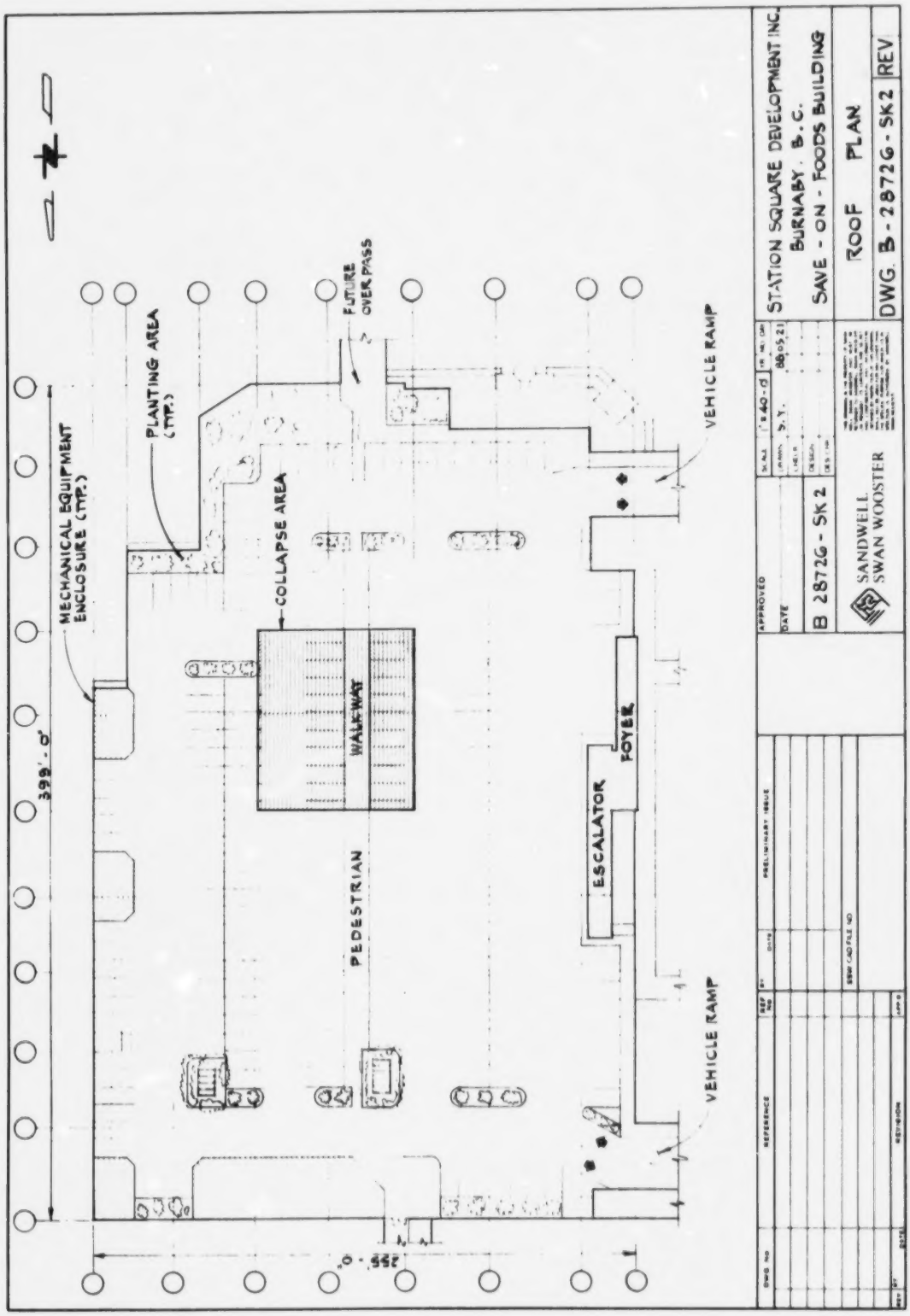
9. REFERENCES

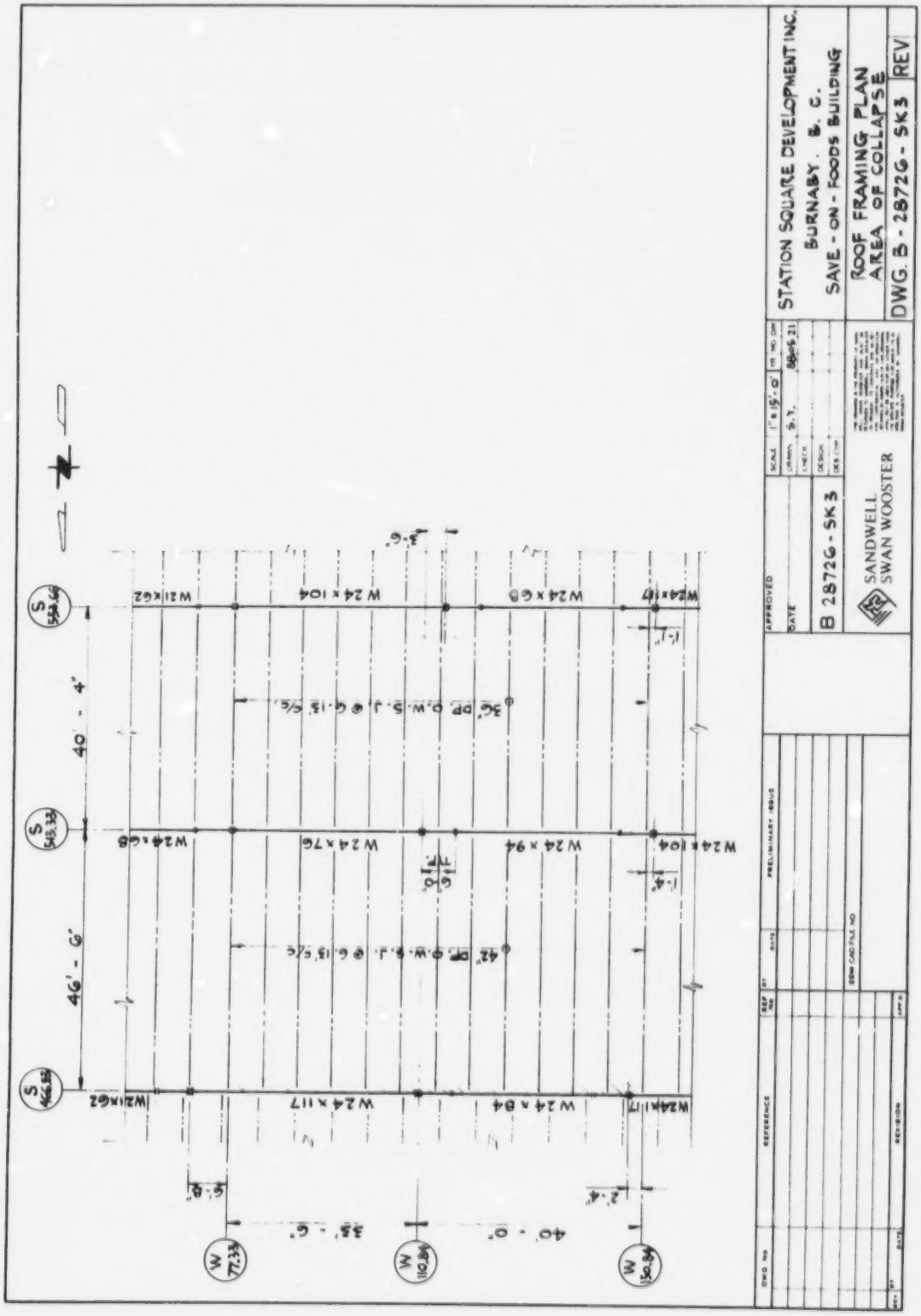
The following documents were used during design analysis:

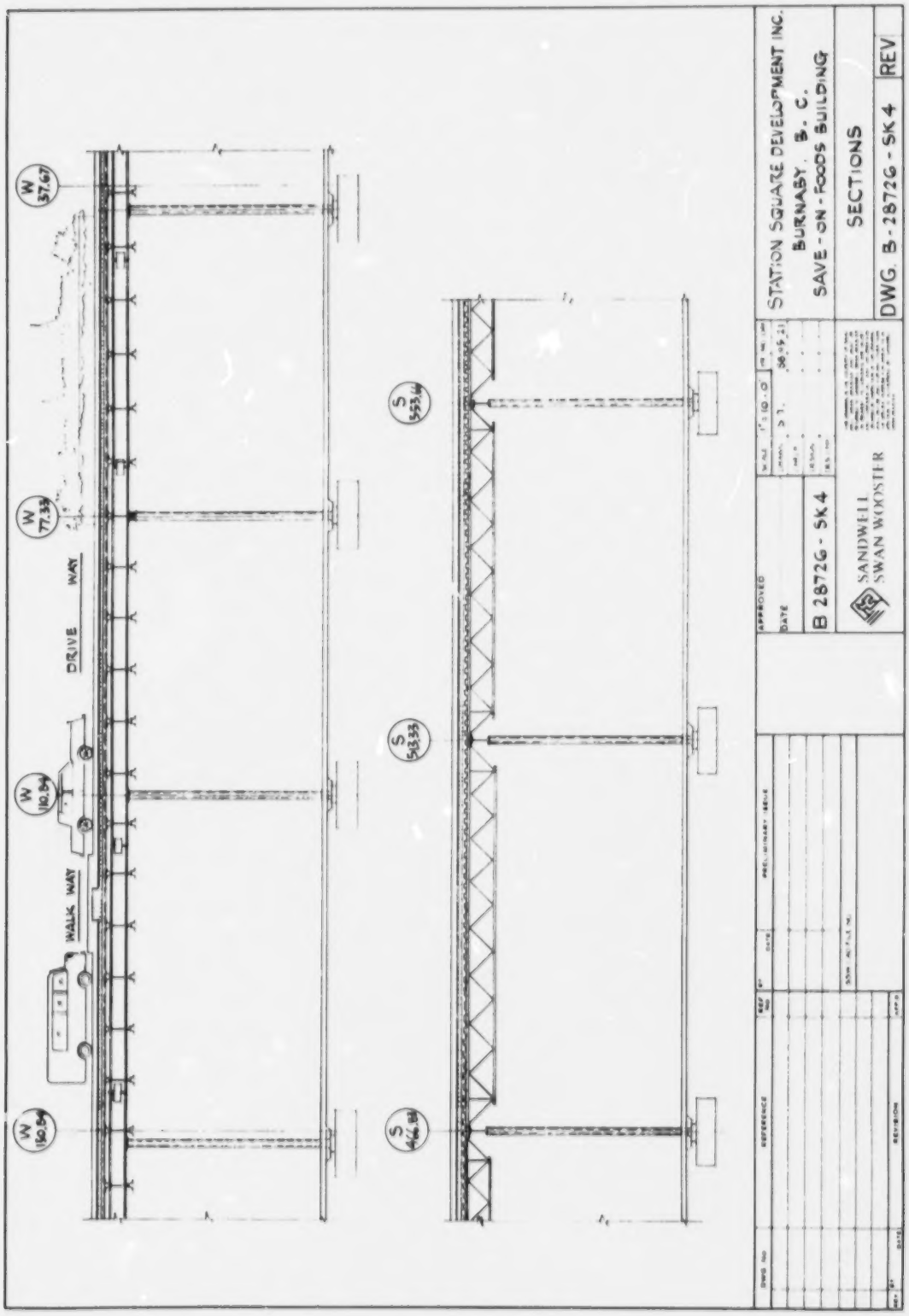
1. The National Building Code of Canada, 1980
2. Supplement to the National Building Code of Canada, 1980
3. The National Building Code of Canada, 1985
4. Supplement to the National Building Code of Canada, 1985
5. The B.C. Building Code
6. Canadian Standard S16 - 1969 "The Design of Steel Structures for Buildings"
7. Canadian Standards CAN3-S16.1-M78 and M84 "Steel Structures for Buildings (Limit States Design)
8. "Handbook of Steel Construction", Fourth Edition, as published by Canadian Institute of Steel Construction
9. "Manual of Steel Construction" Seventh Edition, as published by American Institute of Steel Construction
10. "Structural Steel Designers Handbook" — Frederick S. Merritt — McGraw-Hill.

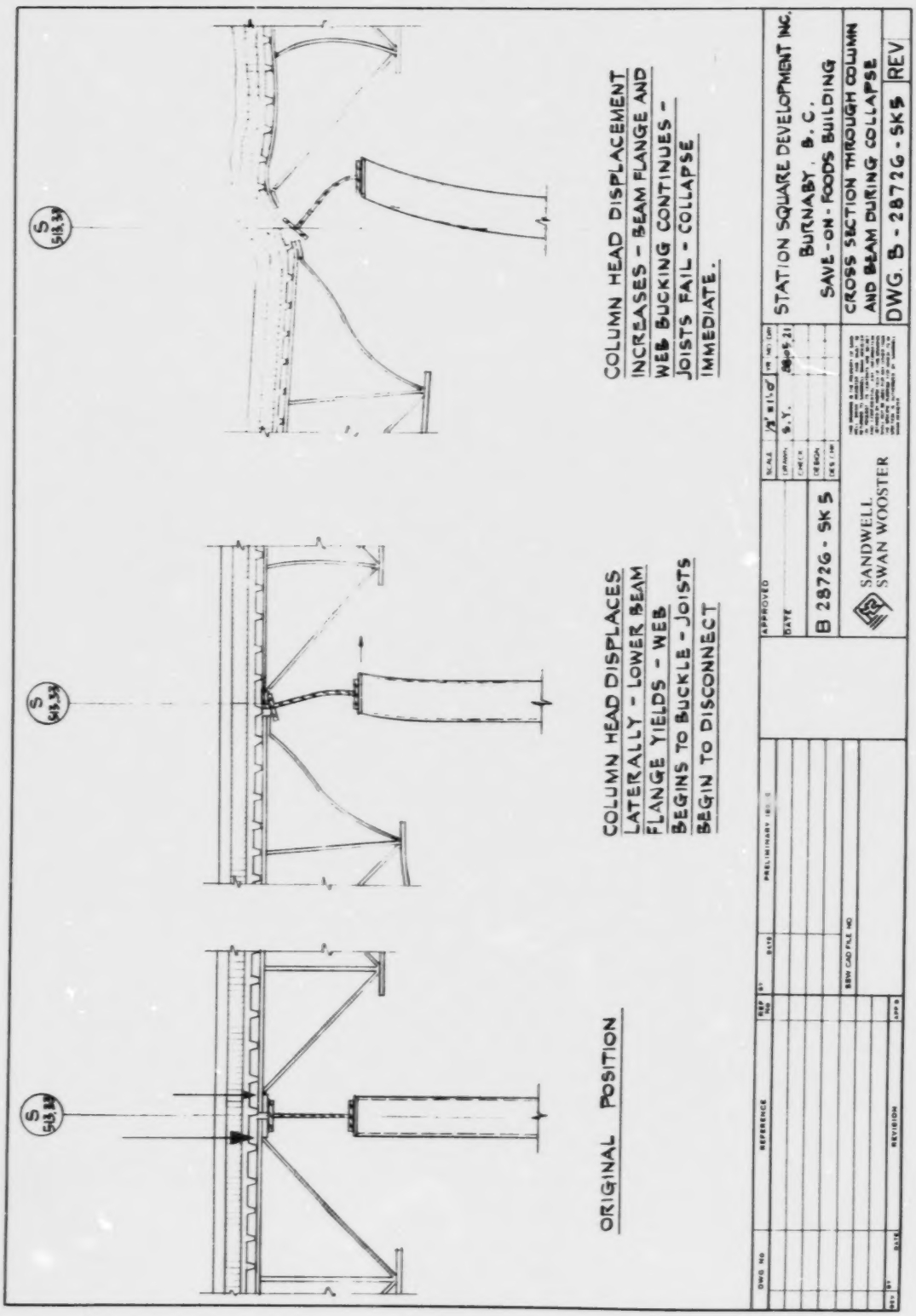
APPENDIX A — Drawings




MECHANICAL EQUIPMENT ENCLOSURE (TYP.)
PLANTING AREA (TYP.)
FUTURE OVER PASS
COLLAPSE AREA
WALKWAY
PEDESTRIAN
ESCALATOR
FOYER
VEHICLE RAMP
VEHICLE RAMP
399' - 0"
STATION SQUARE DEVELOPMENT INC.
BURNABY, B.C.
SAVE - ON - FOODS BUILDING
ROOF PLAN
DWG. B-28726-SK2
REV
B 28726-SK2
SANDWELL
SWAN WOOSTER


STATION SQUARE DEVELOPMENT INC.
BURNABY, B. C.
SAVE-ON-FOODS BUILDING
ROOF FRAMING PLAN
AREA OF COLLAPSE
DWG. B-2872G-SK3 REV
B 2872G-SK3
SANDWELL
SWAN WOOSTER
W24×104
W24×68
W24×76
W24×94
W24×84
W24×117
W21×62
40'-0"
46'-6"
40'-4"
W 77.33
W 110.84
W 150.84


W 37.67
W 77.33
W 110.84
W 150.84
S 553.44
S 513.33
S 466.84
DRIVE WAY
WALK WAY
STATION SQUARE DEVELOPMENT INC.
BURNABY, B. C.
SAVE-ON-FOODS BUILDING
SECTIONS
B 28726 - SK4
DWG. B-28726 - SK4
REV
SANDWELL
SWAN WOOSTER


ORIGINAL POSITION
COLUMN HEAD DISPLACES LATERALLY - LOWER BEAM FLANGE YIELDS - WEB BEGINS TO BUCKLE - JOISTS BEGIN TO DISCONNECT
COLUMN HEAD DISPLACEMENT INCREASES - BEAM FLANGE AND WEB BUCKING CONTINUES - JOISTS FAIL - COLLAPSE IMMEDIATE.
STATION SQUARE DEVELOPMENT INC.
BURNABY, B.C.
SAVE-ON-FOODS BUILDING
CROSS SECTION THROUGH COLUMN AND BEAM DURING COLLAPSE
DWG. B-28726-SK5
REV
B 28726-SK5
SANDWELL SWAN WOOSTER

APPENDIX B — Photographs

APPENDIX C — Test Results

COAST TESTING
1474 DOMINION STREET NORTH VANCOUVER, BRITISH COLUMBIA V7J 1B4 (604) 986-1271
NONDESTRUCTIVE TESTING & MATERIALS ENGINEERING TELEX 04-352570

May 25, 1988
Job # 2133

Sandwell Swan Wooster Inc.
700 - 1190 Hornby Street
Vancouver, B. C.
V6Z 2H6

Attention; Mr. John Rapson, P. Eng.
Civil & Structural Services

Re: Testing of Metrotown Materials

1.0 INTRODUCTION

a) On May 12, 1988 the writer visited Arrow Demolition Inc., Mitchell island, Richmond, B. C. to obtain test samples from one beam and one joist from the Metrotown collapse. The beam and joist were selected by Mr. Rapson. Three and one-half inch wide x 36" long samples were burned from the top and bottom flange and web of the visually-undamaged mid-span region of the beam by Arrow Demolition personnel (Figures 1 and 2), under the writer's direction. Sample removal was witnessed by Mr. Rapson, Mr. Neil Risebrough (Bacon, Donaldson), Mr. Tony Cook (Arrow Demolition), and the writer; also others for varying periods.

. . . 2

COAST TESTING

b) The web was marked longitudinally at 1/4 beam depth (from bottom flange) to indicate the central axis of the test specimen (per CSA Standard G40.20, Appendix A - Attachment 1). The flange samples were marked at 1/3 distance from outer edge to centre to indicate their central axes (Attachment 1).

c) A 20" length was burned from one end of the top chord of the joist for testing (Figure 3).

d) The beam was reportedly specified to be CSA G40.21 grade 44 W steel. We were not informed of the grade of steel specified for the top chord.

. . . 3

COAST TESTING

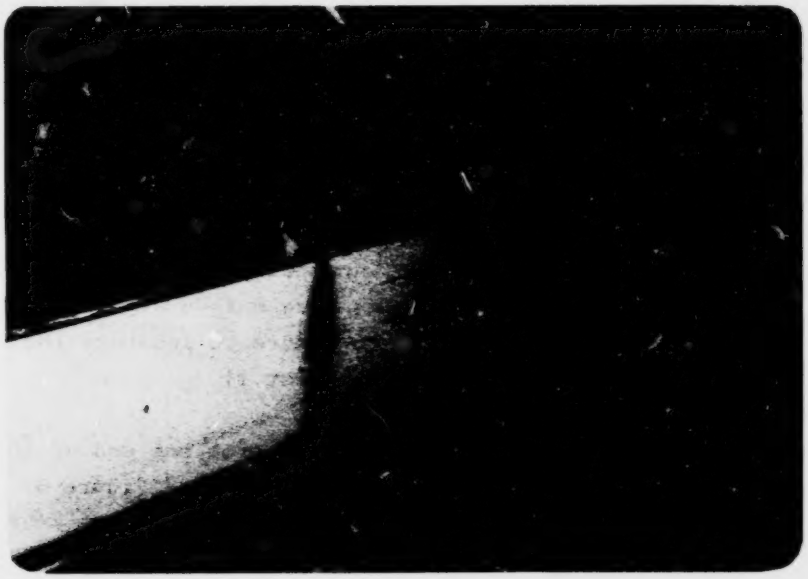


Figure 1: Burning test sample from bottom flange of beam.

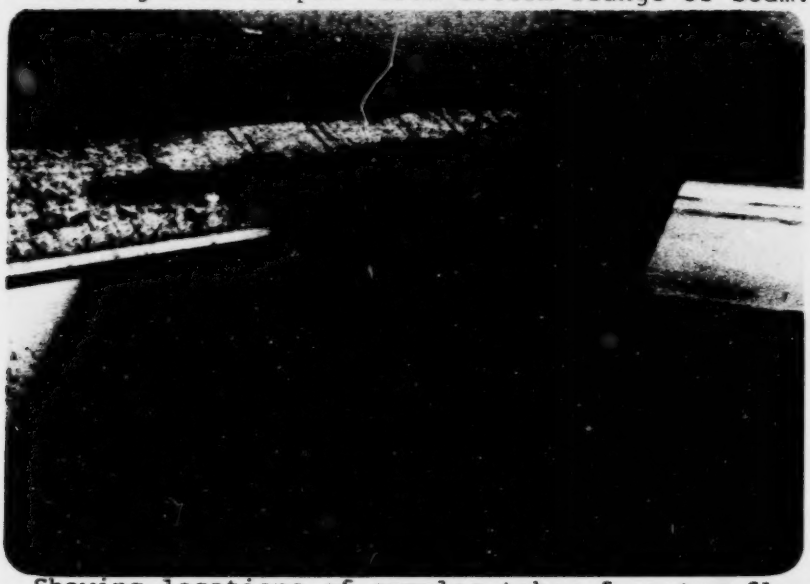


Figure 2: Showing locations of samples taken from top flange and web of beam. Bottom flange sample was taken from other side of beam.

. . . 4

COAST TESTING

Figure 3: Sample was burned from top chord of joist (arrow).

. . . 5

COAST TESTING

2.0 LABORATORY TESTING

2.1 Tensile Tests

a) Tensile test specimens were machined from the top and bottom flange and web samples per CSA G40.20, Figure 1 (Attachment 2). Tensile testing was performed at Coast Testing's laboratory using a Shimadzu Universal Tester, 110,000 lb. capacity. (See Attachment 3 for calibration data.) The tests were witnessed by Mr. Rapson and Mr. Risebrough. The results are shown on pages 6 and 7.

The results for the top and bottom flange and web specimens complied with the requirements of CSA G40.21 Grade 44 W.

. . . 6

COAST TESTING

- 6 -

TENSILE AND GUIDED BEND TEST REPORT

Customer Sandwell Swan Wooster Inc. Purchase Order No.

Address 700-1190 Hornby St., Vancouver, B.C. Date Tested May 17, 1988

Attention Mr. John Rapson Coast Job # 2133

Material Description wide-flange beam, CSA G40.21 - 44W

Specimen Type reduced section tensile per CSA G40.20

Governing Specification

Technician T. Jones & C. Stuart

	SPECIMEN IDENTIFICATION			
	Top Flange	Btm. Flange	Web	
Dimensions				
—Width (in.)	1.573	1.575	1.571	
—Thickness (in.)	0.673	0.657	0.441	
—Area (in^2)	1.059	1.035	0.693	
Yield Load (lb.)	50,000*	50,600*	37,000*	
Yield Strength (ksi)	47.2	48.9	53.4	
—Specified Value (ksi)	44.0min	44.0min	44.0min	
Ultimate Load (lb.)	75,500	75,200	52,700	
Ultimate Strength (ksi)	71.3	72.7	76.0	
—Specified Value (ksi)	65.0-90.0	65.0-90.0	65.0-90.0	
Elongation % in 8"G.L.	27	28	26	
—Specified Value (%)	20min	20min	20min	
Reduction in Area (%)				
Location of Fracture				
Hardness				
Guided Root Bend				
Guided Face Bend				
Guided Side Bend				

Remarks:

* Lower yield

. . . 7

COAST TESTING

- 7 -

Extension Under Load (ins.)	Load (lb.)		
	Top Flange	Bottom Flange	Web
0.001	12,400	20,600	9,000
0.002	25,400	41,500	19,200
0.003	30,900	47,400	32,400
0.004	- -	50,100	37,400
0.005	36,000	50,600	37,400
0.006	43,700	50,600	- -
0.007	- -	50,600	37,200
0.008	47,000	- -	- -
0.009	50,200	50,600	37,100
0.010	50,200	- -	- -
0.011	- -	50,600	37,000
0.012	50,300	- -	- -
0.013	- -	- -	- -
0.014	50,300	- -	- -
0.015	- -	- -	- -

. . . 8

COAST TESTING

b) A tensile test specimen was machined from one flange of the joist sample; the flange without plug welds was used. The test specimen dimensions were per ASTM E8, Figure 6 (Attachment 4). Tensile testing was performed as for the beam specimens. The results follow on pages 9 and 10.

. . . 9

COAST TESTING

TENSILE AND GUIDED BEND TEST REPORT

Customer Sandwell Swan Wooster Inc. Purchase Order No.

Address 700-1190 Hornby St., Vancouver, B.C. Date Tested May 17, 1988

Attention Mr. John Rapson Coast Job # 2133

Material Description Top chord of open-web steel joist

Specimen Type reduced section tensile per ASTM E8

Governing Specification

Technician T. Jones & C. Stuart

	SPECIMEN IDENTIFICATION			
Dimensions				
—Width (in.)	0.500			
—Thickness (in.)	0.500			
—Area (in^2)	0.250			
Yield Load (lb)	15,440*			
Yield Strength (ksi)	61.8*			
—Specified Value (ksi)				
Ultimate Load (lb.)	23,050			
Ultimate Strength (ksi)	92.2			
—Specified Value (ksi)				
Elongation %in 2"G.L.	32			
—Specified Value (%)				
Reduction In Area (%)				
Location of Fracture				
Hardness				
Guided Root Bend				
Guided Face Bend				
Guided Side Bend				

Remarks:

* Lower yield

. . . 10

COAST TESTING

- 10 -

Extension Under Load (ins.)	Load (lb.)
0.001	4,970
0.002	8,730
0.003	12,410
0.004	14,630
0.005	15,220
0.006	- -
0.007	15,440
0.008	- -
0.009	- -
0.010	15,530

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COAST TESTING

2.2 Chemical Analysis

The bottom flange sample from the beam and the top chord sample from the joist were chemically analyzed, as follows:

Sulphur determined by Leco Induction Furnace.
Phosphorus by acid digestion, gravimetric analysis.
Remaining elements by Direct Reading Spectrograph.

The results follow:

Element	Composition, weight %		
	beam sample	CSA G40.21-44W	joist sample
Carbon	0.13	0.22 max.	0.19
Manganese	1.00	0.50-1.50	0.93
Phosphorus	0.008	0.04 max	0.028
Sulphur	0.010	0.05 max	0.039
Silicon	0.03	0.40 max	0.28
Chromium	0.09	- -	0.17
Nickel	0.01	- -	0.08
Molybdenum	0.01	- -	0.03
Vanadium	0.05	V +Nb 0.10 max	<0.01
Aluminum	<0.01	- -	<0.01

<= less than

The composition of the beam was acceptable. The specified composition of the joist was unknown to us.

. . . 12

COAST TESTING

3.0 SUMMARY

The tensile properties and chemical composition of the beam were acceptable.

The top chord of one joist was also tensile and chemically tested; the specifications for this material were unknown to us.

COAST TESTING (CANADA) LTD.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA TREVOR JONES

Trevor Jones, P. Eng.
Metallurgical Engineer

TJ/sj

APPENDIX D — Design Calculations

SANDWELL SWAN WOOSTER

DESIGN SHEET

PAGE 1 OF 23

REFERENCE STATION SQUARE (S.O F) BY JR. DATE 88 04 30

SUBJECT ROOF COLLAPSE CHECKED BG DATE JUNE 22/88 FILE 28726

DEAD LOAD ASSUMPTIONS

(1) UNIT WEIGHT OF CONCRETE = 144 lb/ft^3

i.e 12 lb/ft^2 per inch thickness

(2) TOPPING THICKNESS

Measured on site as between 2.9" and 3.5", with very occasional higher values

Average taken as 3.25" (39 lb/ft^2)

(3) STYROFOAM

Very light, a figure of 0.25 lb/ft^2 per inch thickness assumed. Loading is virtually inconsequential in any case.

(4) STRUCTURAL SLAB

Called as 2½" slab on 1½" deck measured on site between 2½" and 3½" over deck probably due to deck deflection under weight of wet concrete. Average taken as 3" over deck

Supplier's catalogue gives weight of 3" slab over deck as 45.7 lb/ft^2.

46 used in analysis.

(5) JOISTS/BEAMS

42" deep joists to carry ~650 lb/ft weight 12 to 13 lb/ft - at 6.2' spacing say 2 lb/ft^2. Beams at average say 94 lb/ft and 38' spacing equal 2.5 lb/ft^2

Adding for connections, bridging, etc use 6 lb/ft^2.

(6) WATERPROOFING MEMBRANE

Liquid + two sheets protection - say 3 lb/ft^2.

SSW-ENG-012-86

SANDWELL SWAN WOOSTER

DESIGN SHEET

PAGE 2 OF 23

REFERENCE STATION SQUARE (S.O.F) BY JR DATE 88 04 30.

SUBJECT ROOF COLLAPSE CHECKED BCO DATE JUNE 22/88 FILE 28726.

(7) HUNG EQUIPMENT

This includes sprinkler lines, conduit, light fittings, ductwork, and especially hung canopies (such as in photographs).

A blanket allowance of 5 lb/ft² is considered applicable - certain joists should probably be reviewed separately to check that local loadings do not exceed this amount

(8) ROOF TOP

Planter weights can be calculated, as can weight of pedestrian walkway across centre. Mechanical equipment weights are given on drawings.
Collapse area affected by weight of walkway and by planter to the east

(9) VEHICLE LOAD

NBC calls for 2.4 kPa or 50 lb/ft².
Parking bay dimensions set at 18' x 8'-6'
Vehicle weight assumed as 3100 lb and wheelbase as 8'-6" ~~If anything this is light and long, possibly 3100 lb and 8'6" wheelbase should be used, which would slightly increase stresses calculated~~ JR

(10) PEDESTRIAN LOAD

None for analysis at collapse but for final code analysis use 4.8 kPa or 100 lb/ft² over centre 6'-0" of walkway on basis that edges of walkway are blocked by vehicle overhangs.

(11) SNOW LOAD.

By NBC 0.8 x 2.1 = 1.68 kN/m² or 35 lb/ft². ~~Z~~ ~~reduce value to 50% of this plus vehicle load~~ JR

SSW-ENG-012-86

SANDWELL SWAN WOOSTER

DESIGN SHEET

PAGE 3 OF 23

REFERENCE STATION SQUARE (S.O.F) BY JR DATE 88 04 30.

SUBJECT ROOF COLLAPSE. CHECKED BLP DATE June 22/88 FILE 28726

1. TO ANALYSE ROOF LOADING / STRESSES AT TIME OF FAILURE

(a) VEHICLE LOAD

Parking bay dimensions 8'-6" x 18'-0" — 153 S.F.

Vehicle weight varies - average may be reasonably assumed as 3100 lb

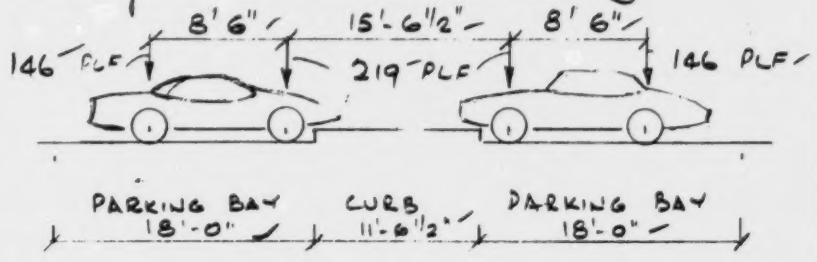
Front wheels were toward central parking curb/slab and it may be reasonably assumed that on average 60% of weight was on front wheels.

∴ Front wheel load 1860 lb / 8'-6" bay — 219 PLF

Rear wheel load 1240 lb / 8'-6" bay — 146 PLF

Wheel positioning - front wheels average say 2'-0" away from curb, wheelbase of vehicles on average 102" / 8'6"

∴ Diagrammatic vehicle loading



SSW-ENG-012-86

SANDWELL SWAN WOOSTER

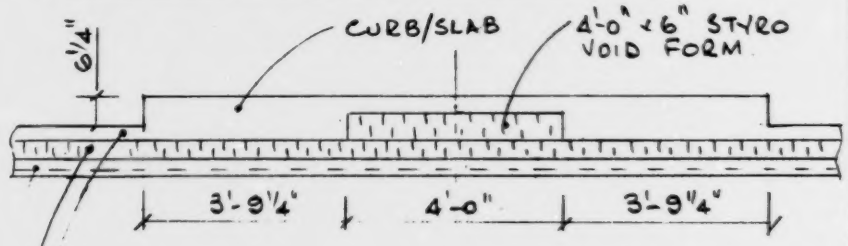
DESIGN SHEET

PAGE 4 OF 23

REFERENCE STATION SQUARE BY JR DATE 88 05 01

SUBJECT ROOF COLLAPSE. CHECKED RCP DATE JUNE 22/88 FILE 28726.

(b) DEAD LOAD AT CURB/SLAB ZONE.



3" NOMINAL, 3 1/4" AV, FIBER/WWF REINFORCED NORMAL WEIGHT CONCRETE WEARING SURFACE

4" BLUE DOW STYROFOAM INSULATION

2 1/2" NOMINAL, 3" AV WWF REINFORCED NORMAL WEIGHT CONCRETE STRUCTURAL SLAB ON KEYED 22 g, 1 1/2" DEEP, 6" SPACING RIBBED METAL DECK WITH MEMBRANE W/PROOFING OVER

UNIT WEIGHTS

Concrete, light reinforcing	12	P.S.F / INCH THICK
Styrofoam	0.25	P.S.F / INCH THICK
Membrane, two ply.	3	P.S.F.
3" cone on 22 g deck	46	P.S.F
Joists + beams	6	P.S.F
Hung equipment allow	5	P.S.F.

SSW-ENG-012-86

SANDWELL SWAN WOOSTER

DESIGN SHEET

PAGE 5 OF 23

REFERENCE STATION SQUARE BY JAR DATE 88 05 01

SUBJECT ROOF COLLAPSE CHECKED [illegible] DATE JUNE 22/88 FILE 28726

DEAD LOADS

General parking area loading
= (3.25 × 12) + (4 × 0.25) + 3 + 46 + 6 + 5 = 100 PSF ✓

Edge portions of curb loading
= [(6.25 + 3.25) × 12] + (4 × 0.25) + 3 + 46 + 6 + 5 = 175 PSF ✓

Centre portions of curb loading
= (3.5 × 12) + (10 × 0.25) + 3 + 46 + 6 + 5 = 104.5 PSF ✓

TRIBUTARY SPANS

Joist span to North = 46'-6" ✓
" " " South = 40'-4" ✓

∴ Span carried by failed beams = $\frac{46.5 + 40.33}{2}$ = 43.42' ✓

Strip →

Loading causes torsion in beam - review later

SSW-ENG-012-86

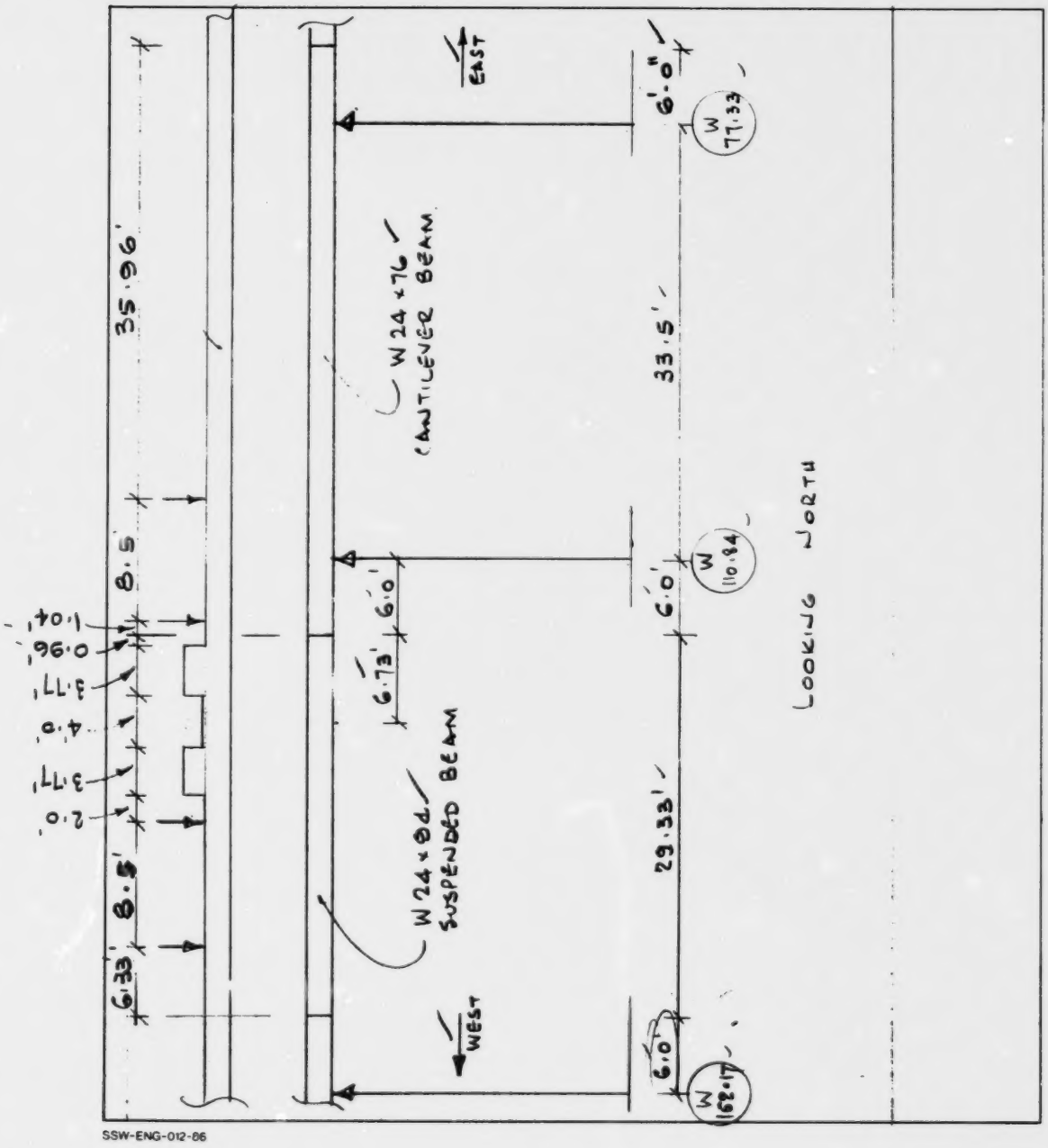
SANDWELL SWAN WOOSTER

DESIGN SHEET

PAGE 6 OF 23

REFERENCE STATION SQUARE BY JAR DATE 88 05 01

SUBJECT Roof COLLAPSE CHECKED EC DATE JUNE 22/88 FILE 28726



SSW-ENG-012-86

SANDWELL SWAN WOOSTER

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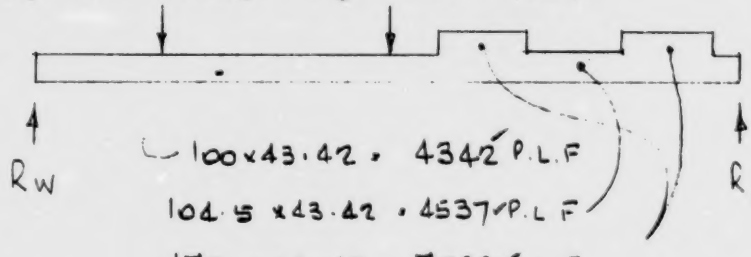
REFERENCE STATION SQUARE BY JAR DATE 88 05 01

SUBJECT ROOF COLLAPSE CHECKED BG DATE June 22/88 FILE 28726.

LOADS ON / REACTIONS OF / MOMENTS / SHEARS IN SUSPENDED BEAM. (W24 x 94)

146 x 43.42 = 6339 ✓ 219 x 43.42 = 9509 ✓

minor corrections marked in blue



100 x 43.42 = 4342 P.L.F

104.5 x 43.42 = 4537 P.L.F

175 x 43.42 = 7598 P.L.F

R_W =	4342 x 0.96 x 0.48	=	2001 ✓
	7598 x 3.77 x 2.845	=	81493 ✓
	4537 x 4.0 x 6.73	=	122136 ✓
	7598 x 3.77 x 10.615	=	304061 ✓
	4342 x 16.83 x 20.92		1528747 528747
	9509 x 14.50 ✓ 92	=	137880 ✓
	6339 x 23.00 ✓	=	145797 ✓
			2,322,115 / 29.33 = 79,172 LB
			2322115

TOTAL BEAM LOAD	4342 x 0.96	=	4168 ✓
	7598 x 3.77	=	28644 ✓
	4537 x 4.0	=	18148 ✓
	7598 x 3.77	=	28644 ✓
	4342 x 16.83	=	73076 73076
	9509	=	9509 ✓
	6339	=	6339 ✓
			168,528
			168528

79172

$\therefore R_E = 168,528 - 79,172 = 89,356$ LB

89

Point of zero shear $= \frac{79252 - 6339}{4342} = 16.79 > 14.83$ ✓

$\therefore$ 14.83' E of R_W. Bending moment at this

point $= 79172 \times 14.83 - 6339 \times 8.5 - \frac{4342 \times 14.83^2}{2} = 642,774$ LB.FT.

642774

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SANDWELL SWAN WOOSTER

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REFERENCE STATION SQUARE BY JAR DATE 88 05 01

SUBJECT ROOF COLLAPSE CHECKED BC DATE June 22/88 FILE 28726.

∴ Bending stress in member W 24 x 94.

$f_b = \pm \frac{M}{S_x} = \frac{642,774 \times 12}{222} = \pm 34,745$ P.S.I. 34745

cf max allowable $0.66 \times 300\ MPa \times 145 = 28,710$

21.0% overstress 21.0%

If we accept mill test for yield of ~ 55,800 PSI then allowable = 36,828 ∴ there is no overstress.

(Test results on W 24 x 76 gave F_u of 47,200 which would mean overstress of 11.5% 11.5%

$47.2 \times .66 = 31.152$ ksi

minor corrections marked in blue

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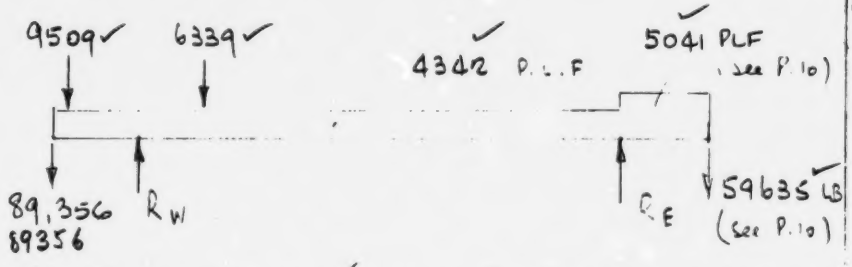
REFERENCE STATION SQUARE BY JAR DATE 88 05 01

SUBJECT ROOF COLLAPSE CHECKED BG DATE June 22/88 FILE 28726.

LOADS ON/REACTIONS OF/MOMENTS/SHEARS IN CANTILEVER BEAM (W24 × 76)

Minor corrections as noted in blue



R_W ↻ + 4342 × 39.5 × 19.75 = 3,387,303

− 5041 × 6.0 × 3.0 = − 90738

× 89,356 × 39.5 = 3,529,562 3529562

9509 × 38.46 = 365.716

6339 × 29.96 = 189.916

− 59635 × 6.0 = − 357.810

7,023,949 / 33.5 = 209,670 LB

7023949 209670

TOTAL LOAD

4342 × 39.5 = 171509

5041 × 6.0 = 30246

× 89,356 = 89,356

9509 = 9509

6339 = 6339

59635 = 59635

366,594 366,594 LB

366594

$\therefore R_E$ = 366,594 − 209,670 = 156,924 LB

156924

Negative B.M over column at R_W.

$= -\left(89356 \times 6 + (9509 \times 4.96) + \left(\frac{4342 \times 6.0^2}{2}\right)\right)$ = 661,457 Lb.FT

89356 661457

$\therefore$ fb at $R_W = \pm \frac{661,457 \times 12}{176}$ = ± 45,100 lb/sq.in

45099

$\therefore$

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DESIGN SHEET

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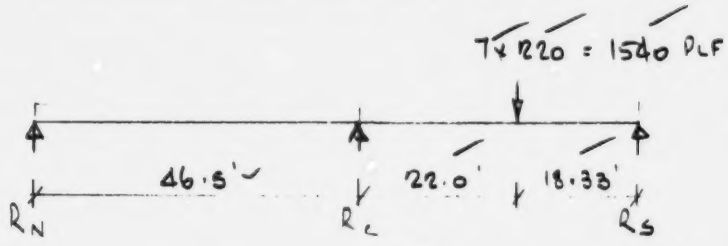
REFERENCE STATION SQUARE BY JAR DATE 88 05 01

SUBJECT ROOF COLLAPSE CHECKED ECP DATE ~~JUNE~~ 22/88 FILE 2870

EFFECT OF PLANTER ON SUSPENDED BEAM TO EAST OF CANTILEVER BEAM.



∴ Reaction on Line C = (100 (46.5 + 40.33)/2 + 1540 x 18.33/40.33 = 5041 PLF

(This compares with 4342 without planter.)

Position of planter is such that 5041 PLF is applied to all of 6'-0" cantilever - all of east suspended beam.

Suspended beam span = 77.33 - 40.67 - 6 - 7 = 23.66'

∴ Reactions = $\frac{5041 \times 23.66}{2}$ = 59,635 lb

Note planter contains ~ 2'-0" of soil, drain rock & bark mulch, which when saturated averages 110 lb/cubic foot. Hence 220 psf over a 7'-0" width, or 1540 plf on the planter ℄.

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REFERENCE STATION SQUARE (S.o.F) BY JAR DATE 88 05 01

SUBJECT ROOF COLLAPSE CHECKED DATE June 22/88 FILE 28726

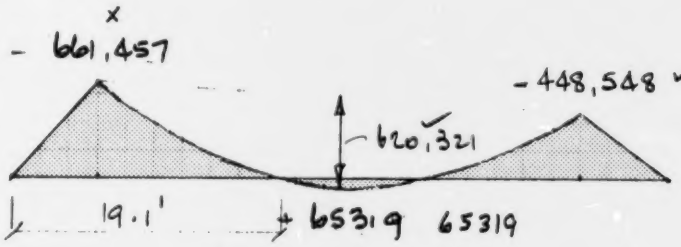
Negative B.M over column at R_E

$$\frac{59635 \times 6 + 5041 \times 6^2}{2} = -448,548 \text{ LB FT}$$

"Free" positive B.M at midspan

$$= \frac{4342 \times 33.5^2}{8} + \frac{6339 \times 3.54 \times 29.96}{33.5} \times \left(\frac{16.75}{29.96}\right) = 620,321 \text{ FT LB}$$

∴ MOMENT DIAGRAM



Beam only just goes into +ve moment at midspan.

This would suggest that the point of contraflexure will not be the position of lateral restraint since tensile stress rate of increase is so small. Assume no restraint until midspan.

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REV'D
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REFERENCE STATION SQ (S.O.F) BY JAR. DATE 89 06 26

SUBJECT ROOF COLLAPSE. CHECKED BG DATE FILE 28726

Check actual f_b against allowable (AISC)

Assumed unbraced length = 1/2 span = 22.75'

Moments at ends virtually zero, large moments within this length, $\therefore C_b = 1.0$.

Then $\frac{L}{r_t} = \frac{22.75 \times 12}{2.29} = 119.2$ Eq 1.5-6b

But $\sqrt{\frac{510 \times 10^3 \times 1.0}{44}} = 107.7 < L/r_t$

F_b allowable $= \frac{170,000 \times 1.0}{(119.2)^2} = 11.96$ ksi

∴ Overstress level = 45.1 / 11.96 377%

To assess column lead restraint - current practice is to provide a force of 1% of flange force. If we assume that this provides a factor of safety of 1.50 over actual buckling lateral force, then potential buckling force at bottom flange over column

$= 45100 \times 8.985 \times 0.682 \times 0.01 / 1.50$ = 1840 lb

If we take column as vertical cantilever, free to translate and rotate, 23'-3" long above floor, then shift under lateral force is

$\Delta = \frac{Pl^3}{3EI} = \frac{1840 \times (23.25 \times 12)^3}{3 \times 29,000,000 \times 485}$ 0.95"

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REFERENCE STATION SQUARE (S.O.F) BY JAR DATE 88 05 04

SUBJECT ROOF COLLAPSE. CHECKED EC DATE JUN 22/88 FILE 28726

If the column was unable to rotate at its head, but free to translate, then

$$\Delta = \frac{P_L^3}{12EI} \checkmark \cdot \frac{1677}{\frac{1840 \times (23.25 \times 12)^3}{12 \times 29{,}000{,}000 \cdot 485}}$$ — 0.24 IN

.21

Likelihood is that actual conditions were somewhere between the two, but trending towards head rotational fixity ✓

∴ Column could have permitted translation of the order of 0.25"

This translation results in increased bending stresses in the lower flange due to the $P\Delta$ effect

$45155 \times .682 \times 8.985 = 276700$ lbs

Moment = Flange force × 0.25 = 69090 lb ✓

S of flange $\left[\right] = \frac{bd^2}{6} \cdot \frac{.682 \times 8.985^2}{6}$ = 9.18 in³ ✓

$$\therefore f_b = \pm \frac{69.090}{9.18}$$ ± 7526 lb/in²

This compressive stress becomes additive to the existing flange stress on the north side, thus leading to an compressive stress of 52,625 lb/in² ✓

i.e exceeds test yield of 48,900 lb/in² ✓

Consider allowable stresses based on flange unsupported length. If we assume unsupported length to be from end of cantilever to midspan, i.e half span

$l = 22'\text{-}9" = 273"$.

Then, from AISC(1970) 1.5.1.4.6(a)

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SANDWELL SWAN WOOSTER

comments marked in blue

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REFERENCE STATION SQUARE (S.O.F) BY JB DATE 88 05 09

SUBJECT ROOF COLLAPSE. CHECKED RG DATE JUNE 22/88 FILE 28726

$$C_b = 1.75 + 1.05\left(\frac{455208}{665134}\right) + 0.3\left(\frac{455208}{665134}\right)^2 \quad C_b = 1$$

$$= 1.75 + 0.72 - 0.14 > 2.3 \text{ — use } 2.3$$

$$\frac{l}{r_t} = \frac{273}{2.32} = 117.7$$ see page 12

$$\therefore F_b = \left[\frac{2}{3} - \frac{55.8 \times 117.7^2}{1530 \times 1000 \times 2.3}\right] 55.8 = 24.96$$

i.e 24,960 lb/sq. in

This is generous, since it assumes that the slight +ve moment at midspan would give beam restraint — thus l is smaller than other analysis would suggest. Also, this uses actual mill test yield

using 44 ksi as F_y

$$F_b = \left[\frac{2}{3} - \frac{44 \times 117.7^2}{1530 \times 1000 \times 2.3}\right] 44 = 21.73 \text{ ksi}$$

i.e 21,730 lb/sq. in.

~~These actual stresses are 1.8 to 2.1 times allowable~~ see page 12

∴ Failure caused by excessive lower flange compressive stress resulting in lateral instability. Column not able to provide restraint & permitted lateral sidesway. Resultant beam displacement increased compressive stress on one side of lower flange leading to yield and immediate flange buckling followed by web rotation & failure.

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SANDWELL SWAN WOOSTER — comments marked in blue — DESIGN SHEET

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REFERENCE STATION SQUARE (S.O F) BY JR DATE 88 06

SUBJECT ROOF COLLAPSE CHECKED ? DATE JUNE 27/88 FILE 28726

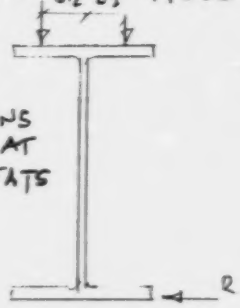
TORSION IN BEAM AT COL HEAD

Reaction (from P 9) — 209.670 lb

Split as follows $\frac{209.670 \times 46.5}{86.83}$ = 112,285 — 209670 see p9 not significant

$\frac{209.670 \times 40.33}{86.83}$ = 97,385

112285 2½' 2½ 97385



T = (112,285 − 97385) × 2.5 — 37,250 lb in

JOIST REACTIONS CONSIDERED AT ℄ JOIST SEATS

If we assume that top of beam is held in place by joists then R at bottom flange $\frac{37250}{24}$ — 1550 lb

This force alone would translate column head by

$\frac{1550 \times (23.25 \times 12)^3}{12 \times 29 \times 10^6 \times 485}$ = 0.20"

This would generate lower flange stresses of :-

$\frac{45100 \times 8.985 \times 0.682 \times 0.20}{9.18}$ — ± 6020 lb/in²

Total compressive stress on north side of lower flange

= 45,100 + 6020 = 51,120 > 48,900

Stress exceeds test yield + well exceeds guaranteed yield

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SANDWELL SWAN WOOSTER

minor corrections shown in blue

DESIGN SHEET REVISED *

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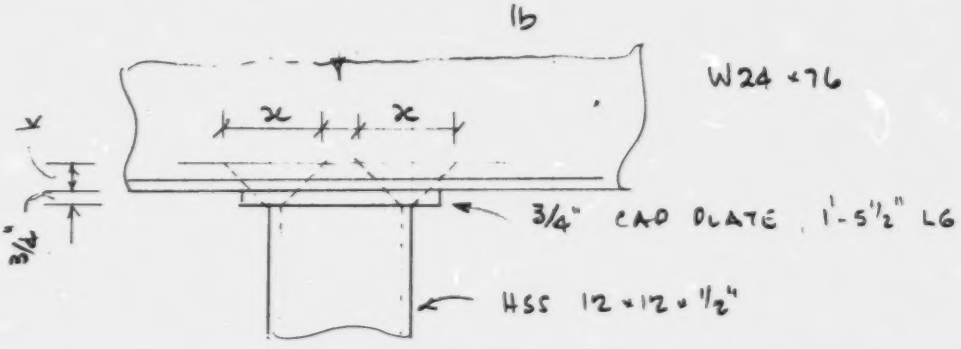
REFERENCE STATION SQUARE (S.O.F) BY JAR DATE 88 05

SUBJECT ROOF COLLAPSE. CHECKED BG DATE June 23/88 FILE 28726

CHECK ON WEB CRIPPLING



Distance $x = 2(k + 0.75) + 0.5 = 2(1.4375 + 0.75) + 0.5 = 4.875"$ ✓

∴ Web area $= 2 \times 4.875 \times 0.44$ ✓ $= 4.29\ in^2$ ✓

∴ Crippling stress $= \frac{209,670}{4.29} = 48,874\ lb/in^2$ (209670)

∴ Web is overstressed in crippling - web probably yields elastically and theoretical load over column wall dimension plus 2 x cap plate thickness which is 13.5" ✓

Web area $[13.5 + (2 \times 1.4375)] \times 0.44 = 7.205\ in^2$ ✓

Crippling stress $= \frac{209,670}{7.205} = 29,100\ lb/in^2$ (209670, 2910)

∴ Even taking a most liberal approach web closely approaches crippling under actual load
Overstress would be severe under design load.

WEB STIFFENERS ARE ESSENTIAL ! ✓

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SANDWELL SWAN WOOSTER

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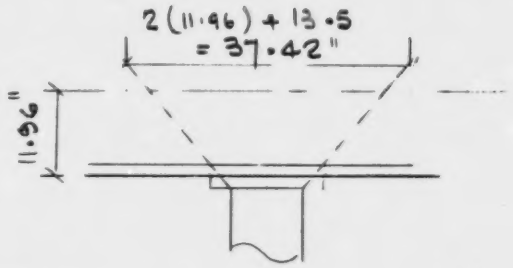
REFERENCE STATION SQUARE (S.F.) BY JAR DATE 88 05

SUBJECT ROOF COLLAPSE CHECKED [illegible] DATE JUNE 23/88 FILE 28726

CHECK ON BEAM WEB AS COLUMN



comments were marked in blue

Effective web area at midheight = $37.42 \times .44$ = 16.46 ✓ in^2

$\therefore f_a = \frac{209,670}{16.46}$ 29670 = 12,738 lb/in^2 ✓

With web as column, fixed ends, $k = 0.7$ — low

$l = 23.92 - (2 \times 1.4375) = 21.045"$ ✓

$r_{44} = \sqrt{\frac{I}{A}}$ $I_{44} = \frac{1 \times 0.44^3}{12}$ = .0071

$A = 1 \times 0.44$ = .44

$\therefore r_{44} = 0.127"$ ✓

$\therefore \frac{kL}{r_4} = \frac{0.7 \times 21.045}{0.127} = 116$ ✓

$\therefore F_a$ from Tables = 11.1 k/in^2 CSA S16 1969 ALLOWABLE STRESS CISC 1970

∴ Web is overstressed in compression, under actual collapse load — design loads would be much worse.

factor of safety is reduced

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comments marked in blue

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REFERENCE STATION SQUARE BY JR DATE 88 06

SUBJECT ROOF COLLAPSE CHECKED [illegible] DATE JUNE 23/88 FILE 28126

? INVESTIGATION ?

FACTORED LOAD DESIGN (CSA S16.1 M78)

From P.7 W24×94 SUSP. BEAM.

load factor should be 1.0 for known service loads i.e. up? moments compared to yield

R_W =	2001 × 1.25	=	2501	
	81493 × 1.25	=	101866	
	122136 × 1.25	=	152670	
	304061 × 1.25	=	380,076	
small error not significant ×	1,528.747 × 1.25	=	1910,934 ×	
	137880 × 1.5	=	206820	
	145797 × 1.5	=	218696	
			2,978,563/29.33	101,383 lb
BEAM LOAD	4168 × 1.25	=	5210	
	28644 × 1.25	=	35805	
	18148 × 1.25	=	22685	
	28644 × 1.25	=	35805	
	73076 × 1.25	=	91345	
	9509 × 1.5	=	14264	
	6339 × 1.5	=	9509	
			214,623	214623 lb

$\therefore R_E$ = 214,623 − 101,383 = 113,240 lb

M_{max} = $101383 \times 14.83 - 9509 \times 8.5 - \frac{5428 \times 14.83^2}{2}$ = 825,796

i.e. 825.8 kip. ft.

M_R of section (CISC Handbook)(1988) = $\phi Z \times F_y$

= $\frac{0.9 \times 254 \times 44}{12}$ = 838 kip ft > 643.96 kft

Approx 1.5% understress, assuming restraint did not reach yield not acceptable by code.

From P.10

Reaction from east end beam = 59635 × 1.25 = 74,544 lb

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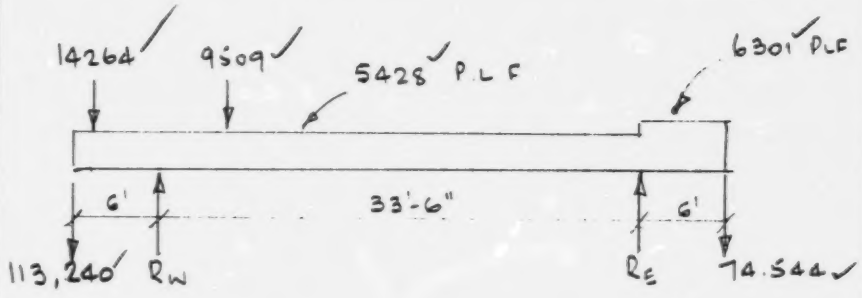
REFERENCE STATION SQUARE BY JR DATE 88 06

SUBJECT ROOF COLLAPSE CHECKED BG DATE June 23/88 FILE 28726

∴ CANTILEVER BEAM - FACTORED LOADS
W24×76

see comments on p 22



Rw + RE	5428 × 39.5 × 19.75	=	4,234,519 ✓
	− 6301 × 6 × 3	=	− 113,418 ✓
	113,240 × 39.5	=	4,472,980 ✓
	14,264 × 39.46	=	548,593 ✓
	9,509 × 29.96	=	284,890 ✓
	− 74,544 × 6.0	=	− 447,264 ✓
			8,980,300/33.5 = 268,070 lb

TOTAL LOAD

5428 × 39.5	=	214,406 ✓
6301 × 6		37,806 ✓
113,240		113,240
14264		14,264 ✓
9509		9,509 ✓
74544		74,544 ✓
		463,769 ✓

∴ R_E = 463,769 − 268,070 = 195,699 lb

Negative moment at Rw

= 113,240 × 6 + (14264 × 4.96) + $\left(\frac{5428 \times 6^2}{2}\right)$ = 847,893 lb.ft

i.e. 847.9 kip.ft ✓

Mom, R_E = 74.544 × 6 + 6301 × $6^2/2$ = 560,682 lb.ft

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SANDWELL SWAN WOOSTER

comments marked in blue

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REFERENCE STATION SQUARE BY JR. DATE 88 06

SUBJECT ROOF COLLAPSE. CHECKED BG DATE June 23/88 FILE 28726

If we take l_u as distance between points of contraflexure – as proposed by Code but questionable due to very small +ve moment at midspan However, as per Code, l_u = 19.1' (5.8m) – see diagram P.11

Then from CISC Handbook (1985) - Beam Selection Tables for W24×76, M_R' = 467 Ft kips

18' → M_r' = 493
20' → M_r' = 442
19' → M_r' = 467

∴ Level of overstress = 849.9/467 = 82%

Even if full lateral restraint provided

$M_R = \phi Z \times F_y = \frac{0.9 \times 200 \times 44}{12}$ = 660 Ft kips ✓

∴ Level of overstress would then have been 849/660 = 29% -

actual calculated load (load factor = 1.0)

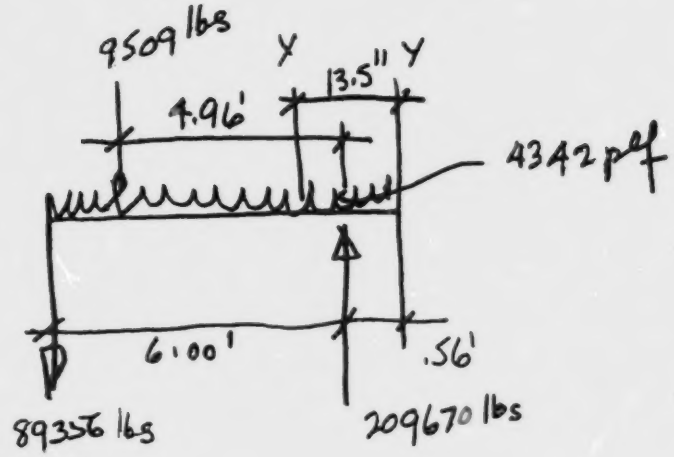
M_f = 661.5 k ft > 581 k-ft

i.e. beam flange in compression is above yielded.

even if resistance factor (ϕ) is disregarded

M_r^* = 646 k ft and beam was loaded to theoretical yield stress

SSW-ENG-012-86 ✓ Test F_y = 48.9 ksi M_y = 717.2 k-ft ϕ = .9 → M_y' = 645.5 k-ft



$$M_y = \frac{4342(6.56)^2}{2} + 9509(5.52) + 89356(6.56) - 209670(.56)$$

$$M_y = 614676 \text{ ft-lbs}$$

SANDWELL SWAN WOOSTER

corrections marked in blue, also see back of page 20

DESIGN SHEET

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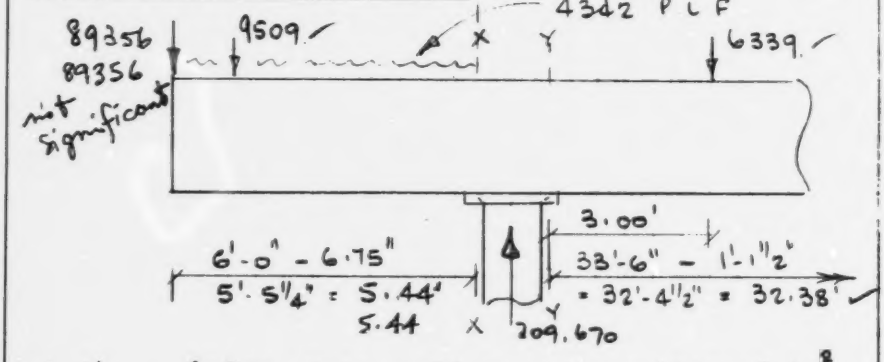
REFERENCE STATION SQUARE BY JR DATE 88 06 11

SUBJECT ROOF COLLAPSE. CHECKED EC DATE June 23/88 FILE 28726

CHECK ON BEAM MOMENTS USING SUPPORT AT COLUMN FACES.

Note: This approach is not considered applicable by CSA S.16.1 - M78, Clause 9.2, but may be given consideration as suggested in the Commentary, Clause 9.1.

SERVICE LOAD ANALYSIS



$\therefore M_x = 89396 \times 5.44 + 9509 \times 4.40 + \frac{4342 \times 5.44^2}{2} = 592,401$ lb.ft

$V_x = 89396 + 9509 + 4342 \times 5.44 = 122,575$ lb.

midspan M for simple beam $= \frac{4342 \times 6.56^2}{2} + 9509 \times 5.52 + 89356 \times 6.56 - 209,670 \times 0.56$ ← Me point of load for simple beam = 614,676 ft lb

see back page 20

$V_Y = 209,670 - 122,575 = 87145$ lb

$\therefore f_b$ at $Y\text{-}Y = \frac{614,676 \times 12}{176} = \pm 41,910$ lb/in^2 X-y = 41910 psi

Still 46% above maximum permissible if lateral restraint were present

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SANDWELL SWAN WOOSTER — DESIGN SHEET

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REFERENCE STATION SQUARE BY JR DATE 88 06 11

SUBJECT Roof COLLAPSE CHECKED SG DATE June 23/88 FILE 28726

Compare with allowable f_b under actual unsupported length, see p.12 =	11,960 lb/in²
∴ 4191.0 / 11,960 gives 350% overstress	X
Furthermore, this approach puts moments into column -	
M_c = (122,525 − 87145) × 6.75" =	238,815 lb·in
∴ f_b in col = 238,815 / 80.9 =	± 2950 lb/in²
Check column	
$\frac{kL}{r}$ = $\frac{0.8 \times 23.25 \times 12}{4.66}$ = 48, ∴ F_a =	24.66 k/in²
f_a = $\frac{209,830}{22.4}$ = 9.367 lb/in² =	9.367 k/in²
∴ $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ = $\frac{9.367}{24.66} + \frac{2.95 \times 1.16}{0.66 \times 50}$ = (.377 + .136 = .513) <	< 1.0
Column will comfortably handle moments However, beam is still substantially overstressed under collapse loads, and would be even more so under design criteria. Worst case design criteria (full live load on drop-in and cantilever, no live load on main span) would increase column moment, but unlikely that column would be incompetent even under this loading.	

SSW-ENG-012-86

SANDWELL SWAN WOOSTER

DESIGN SHEET

PAGE 23 OF 23

REFERENCE STATION SQUARE BY JR DATE 88 06 15

SUBJECT ROOF COLLAPSE CHECKED BC DATE June 23/88 FILE 28726

Consider factored load analysis (S.16.1 - M78)

Ref P.19.

M_{YY} = 113,240 × 6.56 + 14264 × 5.52 + $\frac{5428 \times 6.56^2}{2}$ − 268070 × 0.56 = 788,266

∴ Overstress = 788.3/467 = 69%

If lateral restraint provided, then

Overstress = 788.3/581 = 35.6%

M_{Y-Y} is more critical

SSW-ENG-012-86

SANDWELL SWAN WOOSTER

DESIGN SHEET

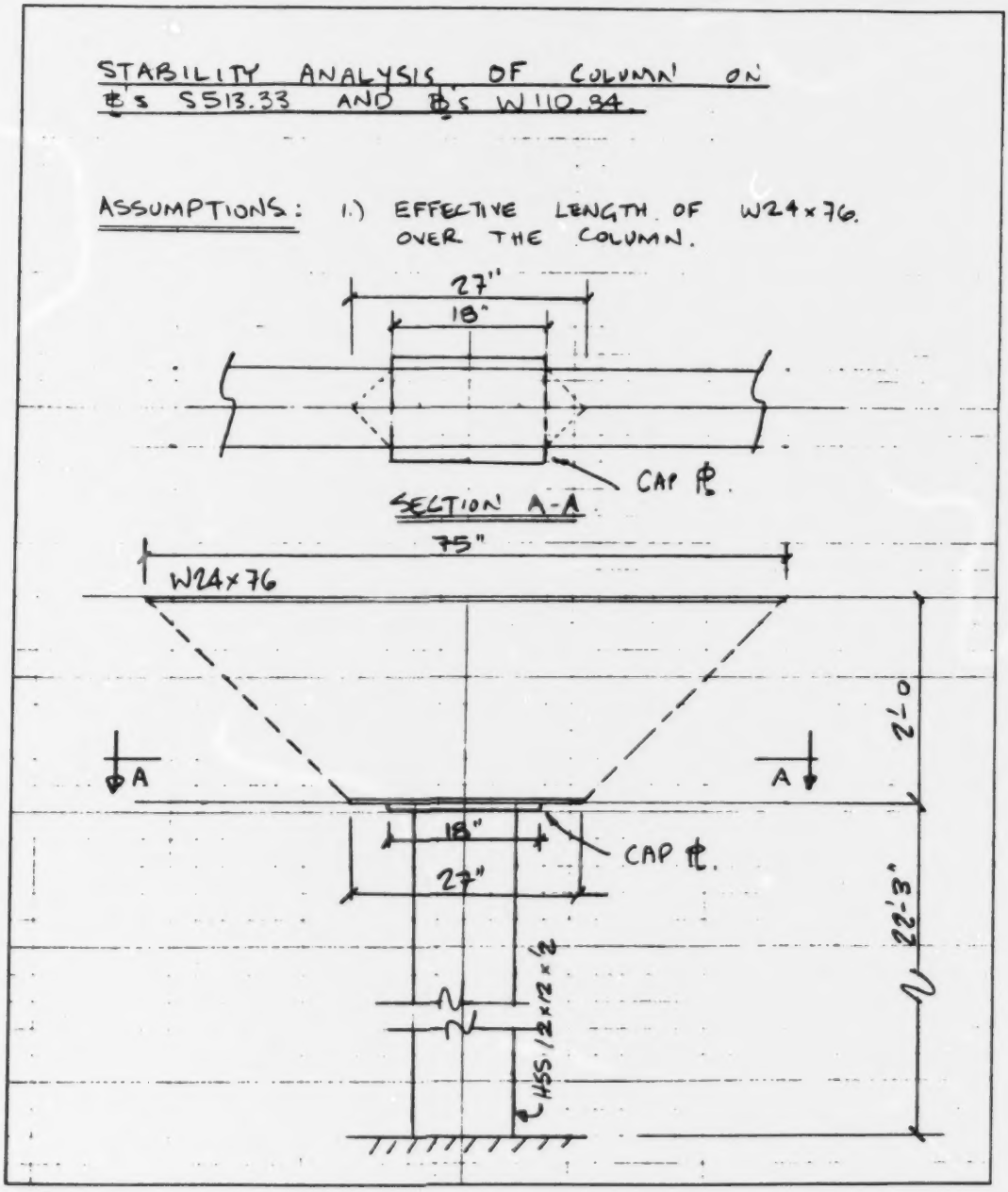
PAGE OF

REFERENCE STATION SQUARE BY P.M. DATE MAY 31/88

SUBJECT ROOF COLLAPSE CHECKED DATE FILE

STABILITY ANALYSIS OF COLUMN ON
℄s S513.33 AND ℄s W110.84.

ASSUMPTIONS: 1.) EFFECTIVE LENGTH OF W24×76.
OVER THE COLUMN.



SSW-ENG-012-86

Sandwell Swan Wooster Inc.
J
G

***** INITIALIZING DATA *****

Job Description: J

Frame Description: G

Structure Parameters

Members	7
Joints	8
Springs	0
Sections	6
Materials	1
Load Cases	1
Load Combinations .	0

Analysis Options

Stability Analysis
Imperial Units

27" BOT. FLG

Stability Analysis Data:

Load Case: 1

Load Factor: 1

Maximum Accuracy: .01
Minimum Diagonal: .001
Maximum Factor: 20
Maximum Iterations: 25
Scale: N

User Name: S

P-FRAME 1.04 (c) 1985

Softek Services Ltd
Head Office:
5729 West Boulevard, Ste 2
Vancouver, B.C. V6M 3W8
Canada (604)263-2726

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FRAME Input Data
S

str No. 30
30 May 88 11:13 am

Sandwell Swan Wooster Inc.
J
S

*** JOINT DATA ***

Joint Number	X - coord. (feet)	Y - coord. (feet)	X - Degree of Freedom	Y - Degree of Freedom	Z - Degree of Freedom
1	0	0	0	0	0
2	0	22.25	1	1	1
3	0	22.75	1	1	1
4	0	23.25	1	1	1
5	0	23.75	1	1	1
6	-.21	24.25	0	1	0
7	0	24.25	1	1	1
8	.21	24.25	0	1	0

Note: Degree of Freedom: 0=restrained 1=free j=coupled to joint 'j'

*** SECTION PROPERTY DATA ***

Sec No.	X-sectional Area (in2)	Mom. Inertia (in4)	Shear Area (in2)	Section Mod (in3)	Plastic Moment Capacity (K-ft)
1	22.4	485	242	0	0
2	14.25	.234	.234	0	0
3	19.8	.319	.319	0	0
4	25.08	.405	.405	0	0
5	30.36	.49	.49	0	0
6	51	1.97	1.97	0	0

Notes:
1. Non-zero Cross-sectional Area and Moment of Inertia are mandatory.
2. For non-zero Shear Area, shear stresses are calculated.
3. For non-zero Shear Area and Shear Modulus, secondary deflections due to shear are included (linear elastic analysis only).
4. For non-zero Elastic Section Modulus (S), stresses are calculated.
5. Non-zero Plastic Moment Capacity is mandatory for plastic analysis.

*** MATERIAL PROPERTY DATA ***

Material Number	Youngmod (ksi)	Shearmod (ksi)	Density (K/ft3)	Coeff Exp (/F 1.E6)	Fy Yield (ksi)
1	29000	12000	0	0	44

[illegible] Input Data Str No. 30
3 30 May 88 11:13 am

Sandwell Swan Wooster Inc.
J
G

Notes:
1. Elastic Modulus (Young s Modulus) is mandatory.
2. For non-zero Shear Modulus and Shear Area, secondary deflections due to shear are included (linear elastic analysis only).
3. Non-zero density is required if self-weight is specified and member weight is to be considered (linear elastic and plastic analysis).
4. Non-zero Thermal Coefficient of Expansion is required for thermal loads. (linear elastic and plastic analysis).
5. Non-zero Yield Stress is mandatory for plastic analysis.

***** MEMBER CONNECTIVITY DATA *****

Member Number	Lower Joint	Greater Joint	Section Number	Material Number	Lower End Type	Greater End Type	Attribute Type	Length (ft)
1	1	2	1	1	1	1	1	22.25
2	2	3	2	1	1	1	1	.5
3	3	4	3	1	1	1	1	.5
4	4	5	4	1	1	1	1	.5
5	5	7	5	1	1	1	1	.5
6	6	7	6	1	1	1	1	.21
7	7	8	6	1	1	1	1	.21

Notes:
1. Member End Types: 1=fixed (rigid connection) 0=pinned (pinned connection).
2. Attribute Type 0 indicates that the member has been deleted.

Sandwell Swan Wooster Inc.
3
6

***** LOAD INITIALIZING DATA *****

Load Case	Number of Loaded Joints	Number of Loaded Members	Load Case Description
1	2	0	L

***** JOINT LOAD DATA *****

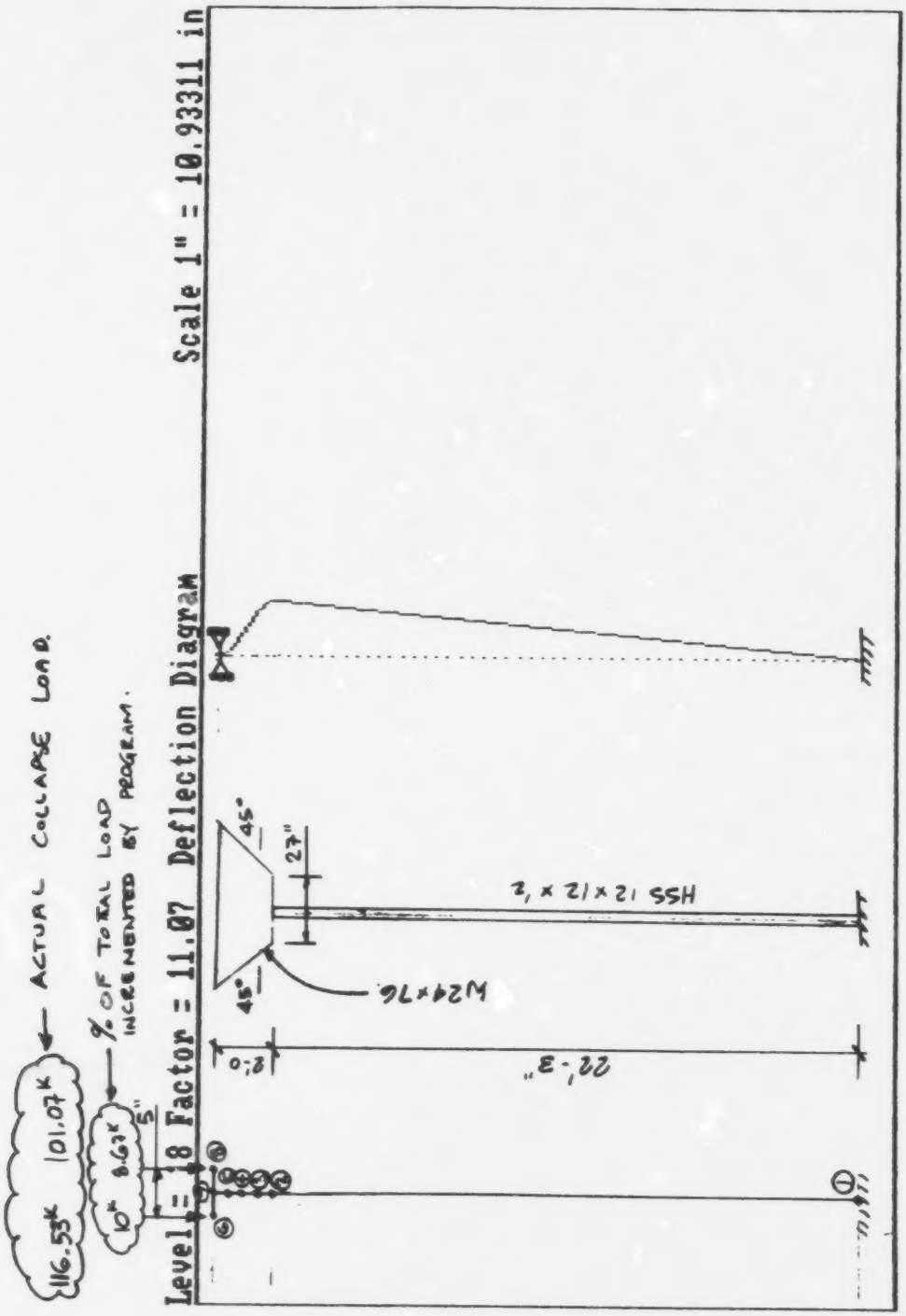
LOAD CASE 1

Record Number	Loaded Joint	Horizontal Load (kips)	Vertical Load (kips)	Torsional Load (K-ft)
1	6	0	-10	0
2	8	0	-8.67	0

Notes:
1. Joint loads act in the global coordinate system.
2. Positive Horizontal Loads act in the positive X direction.
3. Positive Vertical Loads act in the positive Y direction.
4. Positive Torsional Loads act counter-clockwise.

P-FRAME Stability analysis results
S

Str No. 30
30 May 88 09:50 am



P-FRAME Stability analysis results
8

Str No. 30
30 May 88 2:22 pm

Sandwell Swan Wooster Inc.
STATION SQUARE
STABILITY

***** INITIALIZING DATA *****

Job Description: STATION SQUARE

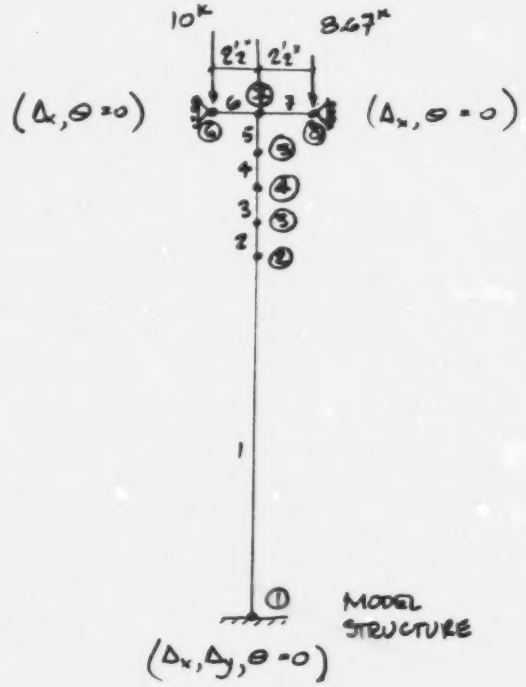
Frame Description: STABILITY

Structure Parameters

Members	7
Joints	8
Springs	0
Sections	6
Materials	1
Load Cases	1
Load Combinations	0

Analysis Options

Stability Analysis
Imperial Units



Stability Analysis Data:

Load Case: 1

Load Factor: 1

Maximum Accuracy: .01
Minimum Diagonal: .001
Maximum Factor: 20
Maximum Iterations: 25
Scale: N

User Name: PM

P-FRAME 1.04 (c) 1985

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Head Office:
5729 West Boulevard, Ste 2
Vancouver, B.C. V6M 3W8
Canada (604)263-2726

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Sandwell Swan Wooster Inc.
STATION SQUARE
STABILITY

***** JOINT DATA *****

Joint Number	X - coord. (feet)	Y - coord. (feet)	X - Degree of Freedom	Y - Degree of Freedom	Z - Degree of Freedom
1	0	0	0	0	0
2	0	22.25	1	1	1
3	0	22.75	1	1	1
4	0	23.25	1	1	1
5	0	23.75	1	1	1
6	-.21	24.25	0	1	0
7	0	24.25	1	1	1
8	.21	24.25	0	1	0

Note: Degree of Freedom: 0=restrained 1=free j=coupled to joint 'j'

***** SECTION PROPERTY DATA *****

Sec No.	X-sectional Area (in2)	Mom. Inertia (in4)	Shear Area (in2)	Section Mod (in3)	Plastic Moment Capacity (K-ft)
1	22.4	485	242	0	0
2	14.52	.234	14.52	0	0
3	19.8	.319	19	0	0
4	25.08	.405	25.08	0	0
5	30.36	.49	30.36	0	0
6	51	1.97	51	0	0

Notes:
1. Non-zero Cross-sectional Area and Moment of Inertia are mandatory.
2. For non-zero Shear Area, shear stresses are calculated.
3. For non-zero Shear Area and Shear Modulus, secondary deflections due to shear are included (linear elastic analysis only).
4. For non-zero Elastic Section Modulus (S), stresses are calculated.
5. Non-zero Plastic Moment Capacity is mandatory for plastic analysis.

***** MATERIAL PROPERTY DATA *****

Material Number	Youngmod (ksi)	Shearmod (ksi)	Density (K/ft3)	Coeff Exp (/F 1.E6)	Fy Yield (ksi)
1	29000	12000	0	0	44

--FRAME Input Data
SM

Str No. 30
31 May 88 09:55 am

Sandwell Swan Wooster Inc.
STATION SQUARE
STABILITY

Notes:
1. Elastic Modulus (Young's Modulus) is mandatory.
2. For non-zero Shear Modulus and Shear Area, secondary deflections due to shear are included (linear elastic analysis only).
3. Non-zero density is required if self-weight is specified and member weight is to be considered (linear elastic and plastic analysis).
4. Non-zero Thermal Coefficient of Expansion is required for thermal loads. (linear elastic and plastic analysis).
5. Non-zero Yield Stress is mandatory for plastic analysis.

***** MEMBER CONNECTIVITY DATA *****

Member Number	Lower Joint	Greater Joint	Section Number	Material Number	Lower End Type	Greater End Type	Attribute Type	Length (ft)
1	1	2	1	1	1	1	1	22.25
2	2	3	2	1	1	1	1	.5
3	3	4	3	1	1	1	1	.5
4	4	5	4	1	1	1	1	.5
5	5	7	5	1	1	1	1	.5
6	6	7	6	1	1	1	1	.21
7	7	8	6	1	1	1	1	.21

Notes:
1. Member End Types: 1=fixed (rigid connection) 0=pinned (pinned connection).
2. Attribute Type 0 indicates that the member has been deleted.

P-FRAME Input Data Str No. 3U
PM 21 May 83 09:55 am

Sandwell Swan Wooster Inc.
STATION SQUARE
STABILITY

***** LOAD INITIALIZING DATA *****

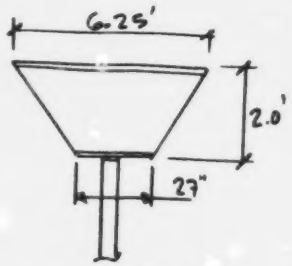
Load Case	Number of Loaded Joints	Number of Loaded Members	Load Case Description
1	2	0	L

***** JOINT LOAD-DATA *****

LOAD CASE 1

Record Number	Loaded Joint	Horizontal Load (kips)	Vertical Load (kips)	Torsional Load (K-ft)
1	6	0	-10	0
2	8	0	-8.67	0

Notes:
1. Joint loads act in the global coordinate system.
2. Positive Horizontal Loads act in the positive X direction.
3. Positive Vertical Loads act in the positive Y direction.
4. Positive Torsional Loads act counter-clockwise.



Sandwell Swan Wooster Inc.
J
6

*** MEMBER FORCES ***

Load Case 1

Mem No.	Load Levl	Axial @ LJ (kips)	Shear @ LJ (kips)	BM @ LJ (K-ft)	Axial @ GJ (kips)	Shear @ GJ (kips)	BM @ GJ (K-ft)
1	1	18.670	.001	.020	-18.670	-.001	.001
	2	37.340	.002	.044	-37.340	-.002	.002
	3	56.010	.003	.073	-56.010	-.003	.003
	4	74.680	.004	.108	-74.680	-.004	.005
	5	93.350	.006	.154	-93.350	-.006	.009
	6	112.020	.008	.215	-112.020	-.008	.014
	7	130.690	.012	.305	-130.690	-.012	.022
	8	149.360	.017	.449	-149.360	-.017	.037
	9	168.030	.026	.728	-168.030	-.026	.068
	10	186.700	.054	1.516	-186.700	-.054	.163
	11	205.370	.773	22.243	-205.370	-.773	2.723
	12	205.557	.885	25.499	-205.557	-.885	3.125
	13	205.743	1.037	29.865	-205.743	-1.037	3.665
	14	205.930	1.251	36.026	-205.930	-1.251	4.427
	15	206.117	1.575	45.375	-206.117	-1.575	5.584
	16	206.303	2.125	61.249	-206.303	-2.125	7.547
	17	206.490	3.266	94.144	-206.490	-3.266	11.616
	18	206.677	7.044	203.067	-206.677	-7.044	25.089
2	1	18.670	.001	-.001	-18.670	-.001	.001
	2	37.340	.002	-.002	-37.340	-.002	.003
	3	56.010	.003	-.003	-56.010	-.003	.004
	4	74.680	.004	-.005	-74.680	-.004	.007
	5	93.350	.006	-.009	-93.350	-.006	.010
	6	112.020	.008	-.014	-112.020	-.008	.015
	7	130.690	.012	-.022	-130.690	-.012	.022
	8	149.360	.017	-.037	-149.360	-.017	.034
	9	168.030	.026	-.068	-168.030	-.026	.057
	10	186.700	.054	-.163	-186.700	-.054	.124
	11	205.370	.773	-2.723	-205.370	-.773	1.897
	12	205.557	.885	-3.125	-205.557	-.885	2.175
	13	205.743	1.037	-3.665	-205.743	-1.037	2.548
	14	205.930	1.251	-4.427	-205.930	-1.251	3.075
	15	206.117	1.575	-5.584	-206.117	-1.575	3.875
	16	206.303	2.125	-7.547	-206.303	-2.125	5.232
	17	206.490	3.266	-11.616	-206.490	-3.266	8.045
	18	206.677	7.044	-25.089	-206.677	-7.044	17.361
3	1	18.670	.001	-.001	-18.670	-.001	.001
	2	37.340	.002	-.003	-37.340	-.002	.003
	3	56.010	.003	-.004	-56.010	-.003	.005
	4	74.680	.004	-.007	-74.680	-.004	.006
	5	93.350	.006	-.010	-93.350	-.006	.007
	6	112.020	.008	.015	-112.020	-.008	.009
	7	130.690	.012	-.022	-130.690	-.012	.009
	8	149.360	.017	-.034	-149.360	-.017	.008
	9	168.030	.026	-.057	-168.030	-.026	.003
	10	186.700	.054	-.124	-186.700	-.054	-.018

S-nn4M6 Stability analysis results
S
Dec 88 (9:50) am

Sandwell Swan Wooster Inc.
J
G

Load Case 1

Mem No.	Load Levl	Axial @ LJ (kips)	Shear @ LJ (kips)	BM @ LJ (K-ft)	Axial @ GJ (kips)	Shear @ GJ (kips)	BM @ GJ (K-ft)
	12	205.557	.885	-2.175	-205.557	-.885	-.779
	13	205.743	1.037	-2.548	-205.743	-1.037	-.918
	14	205.930	1.251	-3.075	-205.930	-1.251	-1.115
	15	206.117	1.575	-3.875	-206.117	-1.575	-1.413
	16	206.303	2.125	-5.232	-206.303	-2.125	-1.920
	17	206.490	3.266	-8.045	-206.490	-3.266	-2.971
	18	206.677	7.044	-17.361	-206.677	-7.044	-6.450
4	1	18.670	.001	-.001	-18.670	-.001	.002
	2	37.340	.002	-.003	-37.340	-.002	.003
	3	56.010	.003	-.005	-56.010	-.003	.004
	4	74.680	.004	-.006	-74.680	-.004	.004
	5	93.350	.006	-.007	-93.350	-.006	.002
	6	112.020	.008	-.009	-112.020	-.008	-.001
	7	130.690	.012	-.009	-130.690	-.012	-.008
	8	149.360	.017	-.008	-149.360	-.017	-.022
	9	168.030	.026	-.003	-168.030	-.026	-.054
	10	186.700	.054	.018	-186.700	-.054	-.152
	11	205.370	.773	.675	-205.370	-.773	-2.863
	12	205.557	.885	.779	-205.557	-.885	-3.289
	13	205.743	1.037	.918	-205.743	-1.037	-3.861
	14	205.930	1.251	1.115	-205.930	-1.251	-4.668
	15	206.117	1.575	1.413	-206.117	-1.575	-5.892
	16	206.303	2.125	1.920	-206.303	-2.125	-7.972
	17	206.490	3.266	2.971	-206.490	-3.266	-12.280
	18	206.677	7.044	6.450	-206.677	-7.044	-26.548
5	1	18.670	.001	-.002	-18.670	-.001	.002
	2	37.340	.002	-.003	-37.340	-.002	.003
	3	56.010	.003	-.004	-56.010	-.003	.002
	4	74.680	.004	-.004	-74.680	-.004	.001
	5	93.350	.006	-.002	-93.350	-.006	-.003
	6	112.020	.008	.001	-112.020	-.008	-.010
	7	130.690	.012	.008	-130.690	-.012	-.022
	8	149.360	.017	.022	-149.360	-.017	-.044
	9	168.030	.026	.054	-168.030	-.026	-.087
	10	186.700	.054	.152	-186.700	-.054	-.213
	11	205.370	.773	2.863	-205.370	-.773	-3.508
	12	205.557	.885	3.289	-205.557	-.885	-4.026
	13	205.743	1.037	3.861	-205.743	-1.037	-4.720
	14	205.930	1.251	4.668	-205.930	-1.251	-5.699
	15	206.117	1.575	5.892	-206.117	-1.575	-7.185
	16	206.304	2.125	7.972	-206.304	-2.125	-9.709
	17	206.490	3.266	12.280	-206.490	-3.266	-14.938
	18	206.677	7.044	26.548	-206.677	-7.044	-32.253
6	1	0.000	-10.000	-1.119	0.000	10.000	-.981
	2	.001	-20.000	-2.238	-.001	20.000	-1.962
	3	.002	-30.000	-3.358	-.002	30.000	-2.942
	4	.002	-40.000	-4.479	-.002	40.000	-3.921
	5						

P-FRAME Stability analysis results
S

Str No. 70
30 May 88 09:50 am

Sandwell Swan Wooster Inc.
J
G

Load Case 1

Mem No.	Load Levl	Axial @ LJ (kips)	Shear @ LJ (kips)	BM @ LJ (K-ft)	Axial @ GJ (kips)	Shear @ GJ (kips)	BM @ GJ (K-ft)
	6	.004	-60.000	-6.724	-.004	60.000	-5.876
	7	.006	-70.000	-7.850	-.006	70.000	-6.850
	8	.008	-80.000	-8.981	-.008	80.000	-7.819
	9	.013	-90.000	-10.122	-.013	90.000	-8.778
	10	.027	-100.000	-11.305	-.027	100.000	-9.695
	11	.386	-110.000	-14.072	-.386	110.000	-9.028
	12	.443	-110.100	-14.342	-.443	110.100	-8.779
	13	.518	-110.200	-14.700	-.518	110.200	-8.442
	14	.625	-110.300	-15.201	-.625	110.300	-7.962
	15	.787	-110.400	-15.956	-.787	110.400	-7.228
	16	1.063	-110.500	-17.228	-1.063	110.500	-5.977
	17	1.633	-110.600	-19.854	-1.633	110.600	-3.372
	18	3.522	-110.700	-28.523	-3.522	110.700	5.276
7	1	0.000	8.670	.979	0.000	-8.670	.841
	2	-.001	17.340	1.959	.001	-17.340	1.682
	3	-.002	26.010	2.939	.002	-26.010	2.523
	4	-.002	34.680	3.920	.002	-34.680	3.362
	5	-.003	43.350	4.903	.003	-43.350	4.201
	6	-.004	52.020	5.886	.004	-52.020	5.038
	7	-.006	60.690	6.872	.006	-60.690	5.872
	8	-.008	69.360	7.863	.008	-69.360	6.702
	9	-.013	78.030	8.865	.013	-78.030	7.521
	10	-.027	86.700	9.908	.027	-86.700	8.299
	11	-.386	95.370	12.536	.386	-95.370	7.492
	12	-.443	95.457	12.805	.443	-95.457	7.241
	13	-.518	95.543	13.161	.518	-95.543	6.903
	14	-.625	95.630	13.661	.625	-95.630	6.421
	15	-.787	95.717	14.414	.787	-95.717	5.687
	16	-1.063	95.803	15.685	1.063	-95.803	4.433
	17	-1.633	95.890	18.310	1.633	-95.890	1.827
	18	-3.522	95.977	26.977	3.522	-95.977	-6.822

Notes:
1. Positive axial forces act in the positive local (member) x direction.
2. Positive shear forces act in the positive local (member) y direction.
3. Positive bending moments act counter-clockwise.

APPENDIX E — Vehicle List and Loading

SANDWELL SWAN WOOSTER

REPORT 28726
SAVE-ON-FOODS STORE
ROOF COLLAPSE

STATION SQUARE DEVELOPMENTS INC.
VANCOUVER B.C.

DATE JUNE 1988

APPENDIX E - VEHICLE LIST AND LOADING

The following lists the twenty vehicles which fell during the collapse, plus one which remained hanging over the edge of the opening.

1987	Acura Integra	1977	Chevrolet Impala
1988	Honda Accord	1964	Pontiac Beaumont
1974	Chevrolet Malibu	1987	Honda Accord
1980	Dodge Diplomat	1988	Chevrolet Cavalier
1987	Plymouth Colt	1983	Oldsmobile Cutlass
1975	Ford LTD	1987	Pontiac 6000
1981	Ford Escort	1981	Oldsmobile Omega
1986	Ford Tempo	1985	Dodge Aries
1973	Pontiac LeMans	1975	Oldsmobile Cutlass
1987	Cadillac Eldorado	1975	Plymouth Valiant

1977 Chevrolet Malibu

The total dry weight of these vehicles, as provided by the Insurance Corporation of British Columbia, was 28,950 kg, or 63,835 lb, for an average of 3,040 lb per vehicle. Allowing for partial fuel and storage within the vehicle, a conservative average weight is 3,100 lb.

The vehicles vary from subcompact to full size, but with a preponderance of compact to mid-size vehicles and with an approximately even distribution between front and rear wheel drive.

For the purposes of analysis, a reasonable assumption is that the average wheelbase of these vehicles is 102 inches or 8 ft 6 in. and that the weight distribution front and rear axles is 60%/40%, given that most vehicles and particularly front wheel drive, carry a preponderance of weight on the front axles.

The tendency of many drivers is, when parking, to drive until the front wheels touch the curb; for analysis purposes the front wheels were assumed to be 2 feet away from the curb on average.

APPENDIX F — Professional Background of Author

SANDWELL SWAN WOOSTER

REPORT 28726
SAVE-ON-FOODS STORE
ROOF COLLAPSE

STATION SQUARE DEVELOPMENTS INC.
VANCOUVER B.C.

DATE JUNE 1988

APPENDIX F - PROFESSIONAL BACKGROUND OF AUTHOR

Mr. Rapson received his professional education at Loughborough College of Advanced Technology (now Loughborough University) in England, graduating in 1959 with an Honours Diploma. After working in the United Kingdom and the Middle East he emigrated to Canada in 1966 and joined Swan Wooster Engineering in 1968. He has been continuously employed by Swan Wooster/Sandwell Swan Wooster since that time.

During the past twenty years he has been involved in the design and project management of many industrial, commercial and institutional building projects in Canada and South East Asia.

Some of the clients with whom he has been associated are General Motors of Canada, the T. Eaton Co., Esso Petroleum, International Nickel Co., Government of Canada, The City of St. Catharines, Neptune Terminals Ltd., the State Railway of Thailand, Hayes-Dana Ltd., Government of Indonesia, and the Eaton-Yale Corporation.

Projects in which Mr. Rapson has played a major design role include the expansion of the T. Eaton warehouse and store in Vancouver, the Bangkok container terminal which contained over 400,000 sq.ft. of warehousing and offices, the rehabilitation of the Matsqui penitentiary following the riot and fire damage, the St. Catharines Public Library, the St. Catharines Memorial Arena, and many plant additions or expansions for industrial clients.

During his career Mr. Rapson has investigated a number of structural failures, including the collapse of a steel warehouse roof at Jordan Wines following heavy rainfall and the failure, under snow drift loading, of 20,000 square feet of roof in a 1.2 million square foot parts facility owned by General Motors of Canada at Woodstock, Ontario.

Mr. Rapson has been a Registered Professional Engineer in the Province of Ontario since early 1967 and in the Province of British Columbia since taking up residence in 1980. While still resident in Ontario he was granted the special Consulting Engineer designation.

APPENDIX V — *Steel Decks for Parking*

GUIDELINES FOR HI-BOND STEEL FLOOR APPLICATION TO PARKING GARAGE CONSTRUCTION

(From the Canadian Sheet Steel Building Institute)

1. Type of Garage

Open garage for passenger car parking only.

2. Design Loads

2.1 Uniformly distributed load 2.4 kPa (50 psf).

2.2 Concentrated load shall be limited to a maximum of 11 kN (2,500 lbs.) spread over an area of 750 mm × 750 mm (2½ ft. by 2½ ft.).

3. Deck Profiles

T-30V Hi-Bond or W938 Inverted Hi-Bond.

4. Concrete

4.1 Minimum 90 mm (3½") concrete cover above deck.

4.2 No chloride additive in concrete shall be allowed.

5. Reinforcing Steel

Use 6 × 6 — % mesh.

6. Corrosion Considerations

6.1 Use Z275 (G-90) galvanizing for protection of steel deck.

6.2 The structural slab shall be protected by a waterproofing membrane to stop water seeping through topping.

6.3 A 50 mm (2") concrete topping shall be provided to protect the waterproofing membrane from wearing. It should be sloped to drain off water and to eliminate any ponding.

6.4 All run-off shall be controlled and drained. Special attention shall be paid to the detailing of the edges of the slab and expansion joints to avoid water in contact with steel.

6.5 Use double layer of reinforcing mesh at support to minimize cracking of the concrete slab.

Note: These guidelines are published by the Canadian Sheet Steel Building Institute.

APPENDIX VI — *Dates of Hearings*

DAY 1 — Monday, May 30, 1988

DAY 2 — Tuesday, June 7, 1988

DAY 3 — Wednesday, June 15, 1988

DAY 4 — Friday, June 17, 1988

DAY 5 — Monday, June 20, 1988

DAY 6 — Tuesday, June 21, 1988

DAY 7 — Thursday, June 23, 1988

DAY 8 — Monday, June 27, 1988

DAY 9 — Thursday, June 30, 1988

DAY 10 — Wednesday, July 6, 1988

APPENDIX VII — *List of Witnesses*

Sponsor Affiliation	*Witness*	*Transcript Page Number*
DAY 2 (Tuesday, June 7, 1988)		
	Annabel McDaniel	13
	Estelle Birch	18
	Elsie Barber	23
	Frank Helden	29
	Walter T. Smallwood	40
	Molly A. Jensen	47
	Evelyn J. Graham	51
	Arthur H. Pastro	55
	Eva N. Howie	63
	William E. Baxter	67
Township of Richmond	Robert V. Switzer, P. Admin.	73
City of Vancouver	Roger V. Hebert, P.Eng.	97
DAY 3 (Wednesday, June 15, 1988)		
Corporation of the District of Burnaby	Mayor William J. Copeland	128
	Ernest Mason, P.Eng.	148
	Trevor Burkitt	156
	Greg Kinch	165
DAY 4 (Friday, June 17, 1988)		
Wesbild Enterprises Ltd.	David Bullus	178
Wesbild Enterprises Ltd.	Dr. Iraj Feiz	178
Overwaitea Foods	James L. Boyson	244
Urban Design Group	Richard Kolodziej	267
Urban Design Group	Paul Chiu	297
DAY 5 (Monday, June 20, 1988)		
Commission Advisor	C. Peter Jones, P.Eng.	303
Commission Advisor	Dr. Noel Nathan, P.Eng.	303
Sandwell Swan Wooster Inc.	John A. Rapson, P.Eng.	362
DAY 6 (Tuesday, June 21, 1988)		
Corporation of the District of Burnaby	Frank R. Mehling, P.Eng.	419
Tamm Tacy & Assoc. Ltd.	Joseph C. Harrison, P.Eng.	437
The MSS Group	Tampion A. London, P.Eng.	497
DAY 7 (Thursday, June 23, 1988)		
Amako Construction Ltd.	David McCall	548
Empire Iron Works Ltd.	Alexander Brown	586
Great West Steel	Gordon Ward-Hall, P.Eng.	614

Sponsor Affiliation	*Witness*	*Transcript Page Number*
DAY 8 (Monday, June 27, 1988)		
The Association of Professional Engineers of British Columbia	Norman A. Johnson, P.Eng.	642
The Association of Professional Engineers of British Columbia	Daniel C. Lambert, P.Eng.	645
	Christopher Quaife, P.Eng.	696
	Michael Kondra, P.Eng.	696
	John Carter Maitland, P.Eng.	716
Poon, Wong & Assoc. Inc.	John L. Wong, P.Eng.	724
	Josef Purger, P.Eng.	753
Applied Science Technologists & Technicians of British Columbia	Derek Squirell, A.Sc.T.	775
DAY 9 (Thursday, June 30, 1988)		
Independent Contractors & Businesses Association of British Columbia	Philip Hochstein	795
Independent Contractors & Businesses Association of British Columbia	George Lockery, P.Eng.	795
International Specialized Risk Management (I.S.R.M.) Ltd.	David V. Pym	822
B.C. & Yukon Territory Building & Construction Trades Council	Len Werden	859
DAY 10 (Wednesday, July 6, 1988)		
Workers	William G. Affleck	878
	Jack I.M. Westcott	888
	Stan Zygarliski	900
	Ivan A. Westcott	906
	Frank Mazzocca	924

APPENDIX VIII — *List of Exhibits*

Exhibit No.	*Description*	*Received From*
DAY 1 — Monday, May 30, 1988		
1	Certified copy of Order-In-Council No. 830 dated May 6, 1988, appointing the Inquiry.	
2	Commissioner's Oath of Office sworn on May 11, 1988.	
3	Extract from the May 19, 1988 edition of the B.C. Gazette, announcing the appointment of the Inquiry and giving notice of the hearing.	
4	Newspaper advertisement announcing the Inquiry and giving notice of the hearing, and list of publications and dates of publications in which ad. appears.	
DAY 2 — Tuesday, June 7, 1988		
5	Structural Plan Review & Inspection Policy.	Permits & Licences Dept. Township of Richmond.
6	Building By-Law No. 3720 and Amendments thereto.	Township of Richmond
7	Vancouver Building By-Law No. 6134.	City of Vancouver
DAY 3 — Wednesday, June 15, 1988		
8	Statement of Mr. Ernie Mason.	Mr. Mason
9	Check-out slip #1, Register 13.	Mr. Mason
DAY 4 — Friday, June 17, 1988		
10	Submission of Iraj Feiz.	Wesbild
11	Submission of David Bulius.	Wesbild
12	Correspondence between Tamm Tacy and Wesbild.	J.J. Camp
13	Correspondence between MSS, Tamm Tacy and Wesbild.	J.J. Camp
14	Construction Contract Station Square and Amako.	R. Campbell
15	Submission Overwaitea.	J. Lepp
16	Submission Urban Design.	D. Brenner
17	Architect's Drawings.	Paul Chiu
DAY 5 — Monday, June 20, 1988		
18	Interim Report on the Cause of the Collapse.	C. Peter Jones and Dr. Noel Nathan
19	Sandwell Swan Wooster Report.	John A. Rapson

Exhibit No.	*Description*	*Received From*
DAY 6 — Tuesday, June 21, 1988		
20	Written submission, Burnaby.	Frank Mehling
21	Design Plans S1.1, S5.1, S5.2, S5.3, S5.4, S5.5.	Frank Mehling
22	Letter of Responsibility, Joe Harrison, May 1/87.	Frank Mehling
23	Letter of Lateral Loadings, Joe Harrison, Sept. 16/87.	Frank Mehling
24	Completion letter, Joe Harrison, April 14/88.	Frank Mehling
25	Clarification letter, Paul Chiu, April 22/88.	Frank Mehling
26	Clarification letter, Joe Harrison, May 4/88.	Frank Mehling
27	Temporary occupancy permit.	Frank Mehling
28	Burnaby Building By-law.	Frank Mehling
29a	Burnaby Building Permit B63557	Frank Mehling
29b	Burnaby Building Permit B63189	Frank Mehling
29c	Burnaby Building Permit B64373	Frank Mehling
30	Architect's Drawings S5.1, S5.2, S5.3, S5.4, S5.5	J.C. Harrison
31	Tamm Tacy Opening Remarks	J.C. Harrison
32	Written submission MSS Group	T.A. London
33	Mill Certificate Algoma Steel	R.W. Hunter
DAY 7 — Thursday, June 23, 1988		
34	Resume of David McCall, Amako.	David McCall
35	Statement of David McCall	David McCall
36	Written submission Amako Construction	David McCall
37	Shop drawings of O.W.S.J.s.	Commission
38	Written submission Empire Iron Works.	Alex Brown
39	Written submission Great West Steel.	Gordon Ward-Hall.
40	Drawing for erection of joists EJ5.3, Rev. 1.	Commission
DAY 8 — Monday, June 27, 1988		
41	Submission of Assoc. of Prof. Engineers.	Dan Lambert
41a	Supplementary material for submission 41.	Norman Johnson
42	Brief on Project Management Practice.	Kondra and Quaife
43	Written submission of J. Carter Maitland.	J. Carter Maitland
44	Written submission Poon, Wong & Assoc. Inc.	John L. Wong
45	Written submission Josef Purger	Josef Purger
46	Written submission — Applied Science Technologists & Technicians of B.C.	Derek Squirell
DAY 9 — Thursday, June 30, 1988		
47	Revision of Exhibit #19 (Rapson Report).	John A. Rapson
48	Written submission Independent Contractors & Businesses Assoc. of B.C.	Philip Hochstein
49	Addendum to ICBA brief (Exhibit #48).	Philip Hochstein
50	Submission — Building Trades Council (BCYTBCTC).	Len Werden

Exhibit No.	*Description*	*Received From*
DAY 10 — Wednesday, July 6, 1988		
51	Statement by Mr. Bill Affleck	BCYTBCTC
52	Statement by Mr. Ivan Westcott Jr.	BCYTBCTC
53	Statement by Mr. Stan Zvgarliski	BCYTBCTC
54	Statement by Mr. Ivan Westcott	BCYTBCTC
55	Statement by Mr. Frank Mazzocca	BCYTBCTC

APPENDIX IX — *List of Submissions*

Bill Affleck
Amako Construction Ltd.
Peter S. Anderson
Applied Science Technologists & Technicians of British Columbia
Architectural Institute of British Columbia
Association of Professional Engineers of British Columbia
B.C. & Yukon Territory Building and Construction Trades Council
City of Vancouver
Coast Testing
Continental Insurance Company
The Corporation of the District of Burnaby
Dowco Consultants Ltd.
Empire Iron Works Ltd.
Great West Steel Industries Ltd.
Independent Contractors & Businesses Association of British Columbia
C. Peter Jones, P.Eng./Dr. Noel Nathan, P.Eng.
M. Kondra, P.Eng. and C. Quaife, P.Eng.
J. Carter Maitland, P.Eng.
Frank Mazzocca
MSS Engineering Ltd.
Overwaitea Foods
Poon, Wong & Associates Inc.
Josef Purger, P.Eng.
John A. Rapson, P.Eng.
Richard A. Smart, P.Eng.
Tamm Tacy & Associates Ltd.
Urban Design Group
Urban Development Institute
Jim Warne, P.Eng.
Wesbild Enterprises Ltd.
Ivan Westcott
Ivan Westcott Jr.
Workers' Compensation Board of British Columbia
Stan Zygarliski

APPENDIX X — *List of Plates*

Plate 1: Beam after initial failure but before final collapse.

Plate 2: Parking deck sidewalk, cars and large portion of the roof crashed to the floor.

Plate 3: Beam in post collapse position with flange buckled and web slipping evident.

Plate 4: The roof system.

APPENDIX XI — *List of Figures*

Figure 1: Tension
Figure 2: Compression
Figure 3: Buckling
Figure 4: Bending
Figure 5: Wide flange sections
Figure 6: The beam-column system: stability
Figure 7: Ranges of load and capacity
Figure 8: Probabilities of load and capacity
Figure 9: Investigation: probabilities
Figure 10: Investigation: probable causes
Figure 11: The sidewalk
Figure 12: The cantilever suspended span or Gerber system
Figure 13: The bending moment diagram
Figure 14: The failed beam: bending probabilities
Figure 15: Station Square failure

APPENDIX XII — *List of Additional Contributors*

Max Anderson, Newport Concrete Products Ltd.

Don D. Bergman, P.Eng., Buckland & Taylor Ltd.

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B. H. Johnston, Alberta Community & Occupational Health.

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Robert Klein, photographer

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State Board of Registration for Professional Engineers and Land Surveyors, Atlanta, Georgia.

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Eve Sugar, American Consulting Engineers Council.

Brooke Sundin, United Food & Commercial Workers Union, Local 1518.

Tosh Suzuki, Workers' Compensation Board of British Columbia.

Alan Toon, P.Eng., National Research Council of Canada, University of British Columbia.

J. H. Walker, H.H. Robertson Inc.

L. C. Ward, P.Eng., The Association of Professional Engineers, Geologists & Geophysicists of Alberta.

David B. Weller, B.C. Construction Association.

Doug Young, VicWest Steel Inc.

Henry Zitco, Workers' Compensation Board of British Columbia.

APPENDIX XIII — *Conversion Factors*

COMMON CONVERSION FACTORS

Item	Imperial – SI	SI – Imperial
Acceleration	1 ft./s^2 = 0.304 8 m/s^2	1 m/s^2 = 3.2808 ft./s^2
Area	1 acre = 0.404 685 6 ha 1 ft.^{2} = 0.092 903 04 m^2 1 in.^{2} = 645.16 mm^2 1 mi.^{2} = 2.589 988 km^2 1 yd.^{2} = 0.836 127 4 m^2	1 ha = 2.471 acres 1 m^2 = 10.764 ft.^{2} 1 mm^2 = 1.55x10^{-3} in.^{2} 1 km^2 = 0.3861 mi.^{2} 1 m^2 = 1.20 yd.^{2}
Capacity (Canadian Legal Units)	1 oz. = 28.413 062 mL 1 gal. = 4.546 090 L 1 pt. = 0.568 261 L 1 qt. = 1.136 522 L	1 mL = 35.2x10^{-3} oz. 1 L = 0.220 gal. 1 L = 1.76 pt. 1 L = 0.880 qt.
Density, Mass	1 lb./ft. = 1.488 16 kg/m 1 lb./yd. = 0.496 055 kg/m 1 oz./ft.^{2} = 305.152 g/m^2 1 lb./ft.^{2} = 4.882 43 kg/m^2 1 lb./in.^{2} = 703.069 6 kg/m^2 1 lb./ft.^{3} = 16.018 46 kg/m^3 1 lb./in.^{3} = 27.679 90 Mg/m^3	1 kg/m = 0.672 lb./ft. 1 kg/m = 2.016 lb./yd. 1 g/m^2 = 3.277x10^{-3} oz./ft.^{2} 1 kg/m^2 = 0.205 lb./ft.^{2} 1 kg/m^2 = 1.42x10^{-3} lb./in.^{2} 1 kg/m^3 = 62.4x10^{-3} lb./ft.^{3} 1 Mg/m^3 = 0.0361 lb./in.^{3}
Force	1 kip = 4.448 222 kN	1 kN = 0.225 kip
Length	1 ft. = 0.304 8 m = 304.8 mm 1 in. = 25.4 mm 1 mile = 1.609 344 km 1 yd. = 0.914 4 m	1 m = 3.28 ft. 1 mm = 0.0394 in. 1 km = 0.622 mi. 1 m = 1.09 yd.
Mass	1 lb. = 0.453 592 37 kg 1 ton (2000 lb.) = 0.907 184 74 Mg	1 kg = 2.20 lb. 1 Mg = 1.10 ton = 2200 lb.
Mass per Unit Area	1 lb./ft.^{2} = 4.882 43 kg/m^2	1 kg/m^2 = 0.205 lb./ft.^{2}
Mass per Unit Length	1 lb./ft. = 1.488 16 kg/m	1 kg/m = 0.672 lb./ft.
Moment of Inertia a) Second Moment of Area b) Section Modulus	1 in.^{4} = 416 231.4 mm^4 1 in.^{3} = 16 387.064 mm^3	1 mm^4 = 2.4x10^{-6} in.^{4} 1 mm^3 = 0.061x10^{-3} in.^{3}
Pressure or Stress	1 ksi = 6.894 757 MPa 1 psf = 47.880 26 Pa 1 psi = 6.894 757 kPa	1 MPa = 0.145 ksi 1 Pa = 0.0209 psf 1 kPa = 0.145 psi
Torque or Moment of Force	1 ft.•kipf = 1.355 818 kN•m	1 kN•m = 0.738 ft.•kipf
Volume	1 in.^{3} = 16 387.064 mm^3 1 ft.^{3} = 28.316 85 dm^3 1 yd.^{3} = 0.764 555 m^3	1 mm^3 = 0.061x10^{-3} in.^{3} 1 dm^3 = 0.0353 ft.^{3} 1 m^3 = 1.308 yd.^{3}
Costs	1 $/ft. = 3.28 $/m 1 $/ft.^{2} = 10.764 $/m^2 1 $/yd.^{2} = 1.20 $/m^2 1 $/ft.^{3} = 35.34 $/m^3 1 $/yd.^{3} = 1.307 $/m^3	1 $/m = 0.305 $/ft. 1 $/m^2 = 0.0929 $/ft.^{2} 1 $/m^2 = 0.836 $/yd.^{2} 1 $/m^3 = 0.0283 $/ft.^{3} 1 $/m^3 = 0.765 $/yd.^{3}

APPENDIX XIV — *The Commissioner*

- Professional Administrator
  Chartered Institute of Secretaries
- Senior Certificate
  Department of Municipal Affairs, UBC
- January 1985–August 1987
  Member of B.C. Labour Relations Board
- March 1972–April 1984
  Municipal Manager, Municipality of Surrey
- January 1969–March 1972
  Chief Inspector — Permits and Licence Department, Municipality of Surrey

APPENDIX XV — *Glossary*

Abbreviations
Arm
Axial Load
Beam
Bell Curve
Bending Moment
Bending Moment Diagram
Buckling
Cantilever
Capacity Reduction Factor
Column
Compression
Crippling
Dead Load
Deflection
Factor of Safety
Factored Load
Flange
Force
Force - Reaction
Kip
Lever Arm
Limit States Design
Live Load
Load, Axial
Load, Dead
Load Factor
Load, Factored
Load, Live
Load, Service
Load, Working
Maximum Permissible Stress
Member
Moment
Moment Curve
Point of Inflection
Reaction
Service Load
Shear
Simply Supported Beam
Stress
Structural Steel Section
Tension
Units
W Section
Web
Work
Working Load
Working Stress Design
Yield Strength
Yield Stress

Abbreviations:

M_p = The probable Moment capacity of a member.
M_r = M_p multiplied by a capacity reduction factor.
psi = Pound(s) per square inch.
psf = Pound(s) per square foot.

Arm:

See Moment.

Axial Load:

See Load, Axial.

Beam:

A structural member to carry loads across a horizontal gap by bending.

Bell Curve:

Or probability curve.

The probability of an event is the ratio of the number of favourable chances of its occurrence to the total number of chances both favourable and unfavourable.

If the probability of some quantity, such as the moment capacity of a W24 × 76, having a given value, is plotted vertically against the range of values, the resulting curve has a bell-shape if the causes of variation are random.

Bending Moment:

The external moment at any section of a beam or other flexural member of all the forces which act on the beam on one side of that section.

The internal moment created by the tension and compression stresses in the beam tending to resist the external moment.

Bending Moment Diagram:

Diagram showing the variation of bending moment along the span.

Buckling:

A column or other compression element is said to buckle if it loses its stability and bends sideways. It is a type of failure associated with slender members. Buckling is not necessarily initiated by material failure.

Cantilever:

A beam supported only at one end.

Capacity Reduction Factor:

A factor, less than 1.0, used in limit states design to provide a margin of safety. The probable capacity is reduced by multiplying by this factor.

Column:

A vertical structural member carrying a compressive load.

Compression:

An external force which tends to push parts of a body together is a compression force or load.

An internal stress which tends to keep a part of a body from being pushed together is a compression stress.

Crippling:

Is a form of buckling or folding in a concertina fashion of a local portion of the web of a W section that occurs due to a concentrated force caused by a load or reaction acting in the plane of the web.

Dead Load:

See Load, Dead.

Deflection:

The flexural displacement of a structural member, particularly the vertical displacement of a horizontal beam.

Factor of Safety:

Factor used in working stress design to provide a margin of safety against collapse or serious structural damage. See also load factor, capacity reduction factor, maximum permissible stress, and working stress design.

Factored Load:

See Load, Factored.

Flange:

See Structural Steel Section.

Force:

An action of one body upon a second body which changes or tends to change the state of motion or rest of the second body. Forces are measured in pounds or kips.

Force - Reaction:

In a building or any structure intended to be at rest, each force is balanced by an equal and opposite force called a reaction.

Kip:

1,000 pounds.

Lever Arm:
See Moment.

Limit States Design:
The object of structural design is to ensure that the structure exhibits no undesirable behaviour. "Undesirable behaviour" may include, for example, minor cracking, vibration, excessive deflections, component failure, or complete collapse. Each of these is called a "Limit State," and the object is to reduce the probability of their occurrence to an acceptable value; a relatively high probability of minor cracking may be acceptable, but a very low probability of complete collapse. As is discussed in Chapter 2, "Structures—General Background," probabilities are introduced into the calculations by means of load factors and capacity reduction factors. For example the load factors for minor cracking are 1, whereas higher values are used for other limit states.

Live Load:
See Load, Live.

Load, Axial:
A load acting along the centre of the long dimension of a member.

Load, Dead:
The weight of all permanent structural and other building components carried by a structural element.

Load Factor:
Used in limit states design. The number greater than one by which the design load is multiplied to provide a margin of safety. For dead load, the load factor is 1.25; for live load, the load factor is 1.50.

Load, Factored:
The product of the design load multiplied by the load factor.

Load, Live:
Loads other than dead load, including those due to occupancy, snow, rain, etc.

Load, Service:
The actual dead and live load that the structure is designed to carry. It is synonymous with working load.

Load, Working:
See Working Load.

Maximum Permissible Stress:
Used in working stress design, the greatest stress permissible in a structural member under the action of the service loads. It is usually defined as (limiting strength of material) divided by (factor of safety).

Member:
An individual element of a structure such as a beam or column.

Moment:
Of a force is its tendency to cause rotation about a point. Moment is the product of that force and the perpendicular distance between its line of action and the point. This distance is the "arm" or "lever arm" of the force with respect to the point. If a force of one kip acts with an arm of 6 feet, the moment is 6 Ft.-Kips.

Moment Curve:

If the bending moment is plotted at each point in the length of a beam, the graph so created is a moment curve. The plot is said to be positive (greater than zero) where the curvature of the beam is concave upward and negative (less than zero) where concave downward. Where it changes sign is called the point of inflection.

Point of Inflection:

See Moment Curve.

Reaction:

A downward load on a beam or frame produces an equal upward reaction from its support.

Service Load:

See Load, Service.

Shear:

In a beam, shear is the tendency, at any section or cut along its length, for the left portion to move vertically relative to the right position. Shear equals the algebraic sum of the loads and reactions, (i.e., all the forces) on one side of that section.

Simply Supported Beam:

A beam freely resting on two supports, and not joined to the adjacent structure, as opposed to built-in or continuous beams.

Stress:

An internal force created when a body is subjected to external forces. Usually measured as unit stress, i.e. force per unit area. A force of 100 pounds acting on an area of five square inches produces a unit stress of 100 divided by 5 or 20 pounds per square inch or 20 psi.

Structural Steel Section:

Heated structural steel blocks are fed through rollers to create shapes or sections that efficiently perform their intended function. Two types of section are discussed in this report:

W sections — a shortened designation for wide flange. This type has, when used as a beam, upper and lower horizontal elements called flanges connected by a thin central vertical element called the web. A cross section of a W beam looks like an H on its side. A W section is an efficient beam having a high capacity for moment.

Hollow square sections — this type is efficient in carrying axial loads, having an equal resistance to buckling in all four directions. It is commonly used for columns.

Tension:

An external force which tends to pull a body apart is a tension force or load.

An internal force or stress which tends to keep a body from being pulled apart is a tension stress.

Units:

Modern engineering uses S.I., the international system of units of measurement. Because most readers of this document grew up with the Imperial or English system of pounds, feet, inches, etc., these units have been used exclusively in this report, including this Glossary. For a conversion table, see Appendix XIII.

Web:

See Structural Steel Section.

Work:

The product of force times its displacement. If a force of 10 pounds moves an object two feet, it does 10×2 or 20 foot pounds of work.

Working Load:

Service load.

W Section:

See Structural Steel section.

Working Stress Design:

A design procedure whereby the stresses caused by expected service loads are limited to the failure stress of the material divided by a factor of safety.

Yield Strength:

Strength of a member when the maximum stress reaches yield stress.

Yield Stress:

The stress at which there is a great increase in deformation without an increase in stress. Below this stress, deformation is proportional to stress.

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